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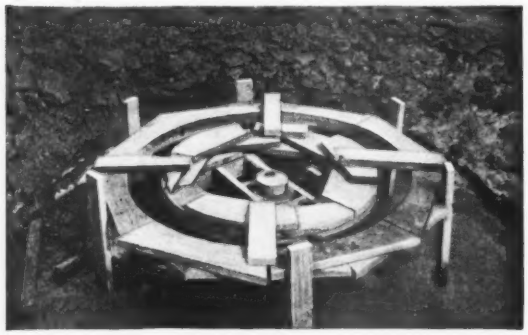

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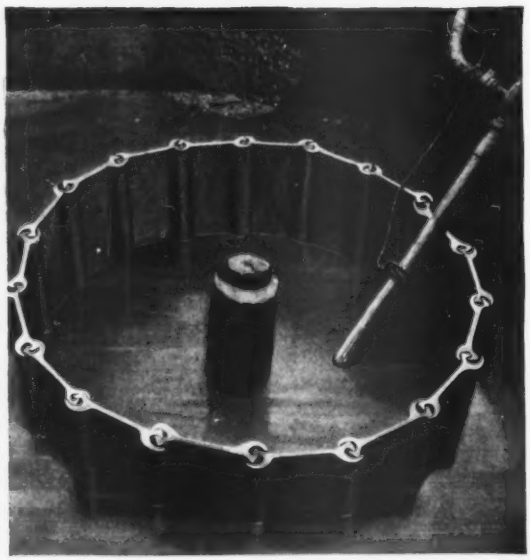

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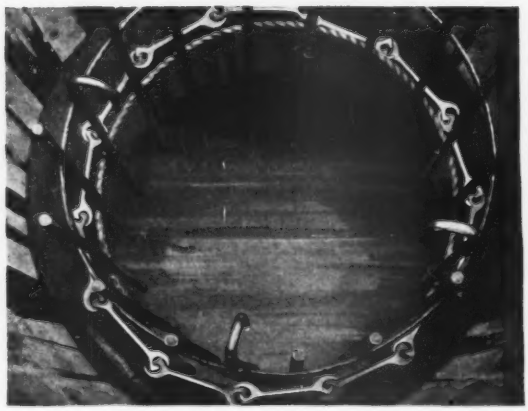

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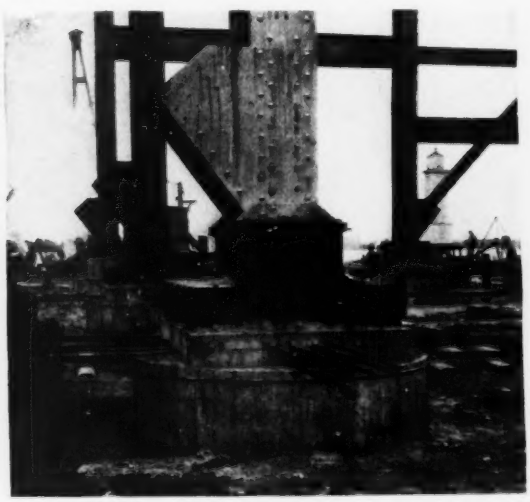

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VOLUME 76 NOVEMBER 9, 1916 NUMBER 19

Jacking Up a Viaduct; Buckling Columns Give Trouble

Settlement of the pile foundations under a steel viaduct carrying Riverside Drive across Manhattan Valley in New York City made a very difficult repair job necessary. The work has been carried out successfully, however, and at very low cost, by the Bureau of Highways, Borough of Manhattan, under E. W. Stern, Chief Engineer, and Robert A. MacGregor, Assistant Engineer. A remarkable phenomenon in the course of the job was incipient failure of the lower portions of the columns, as shown in large distortions and web-buckling.

The Riverside Drive viaduct was completed in 1900. A fairly full description of the structure in *Engineering News* of June 8, 1899, pp. 358 to 360, with a double-page inset of drawings, may be consulted with regard to the details. While that article stated that the work was constructed under the Bureau of Highways, with F. S. Williamson as Consulting Engineer, the fact appears to be that a separate organization over which the Highway Bureau had no jurisdiction was in charge of the design and construction, except that the formal countersign of the Bureau of Highways had to be affixed to vouchers. None of the present officials of the bureau were connected with the work, and little or no detail information is found in the files of the Bureau.

The viaduct consists of plate-girder spans about 65 ft. long and one plate-girder span over Manhattan St. about 128 ft. long. In place of diagonal bracing longitudinally and transversely, there is arch bracing. According to the designers' intentions, this arch work was intended to support part of the moving load, or, in other words, to reduce the deflection of the respective plate girders under moving load, but not under dead-load, the arch and girder being merely in contact under full dead-load conditions. The probabilities are that the arches serve principally as ornaments and as bracing, although the bracing effect (in longitudinal direction at least) is seriously reduced by the inadequacy of the bolt connections between arch and girder.

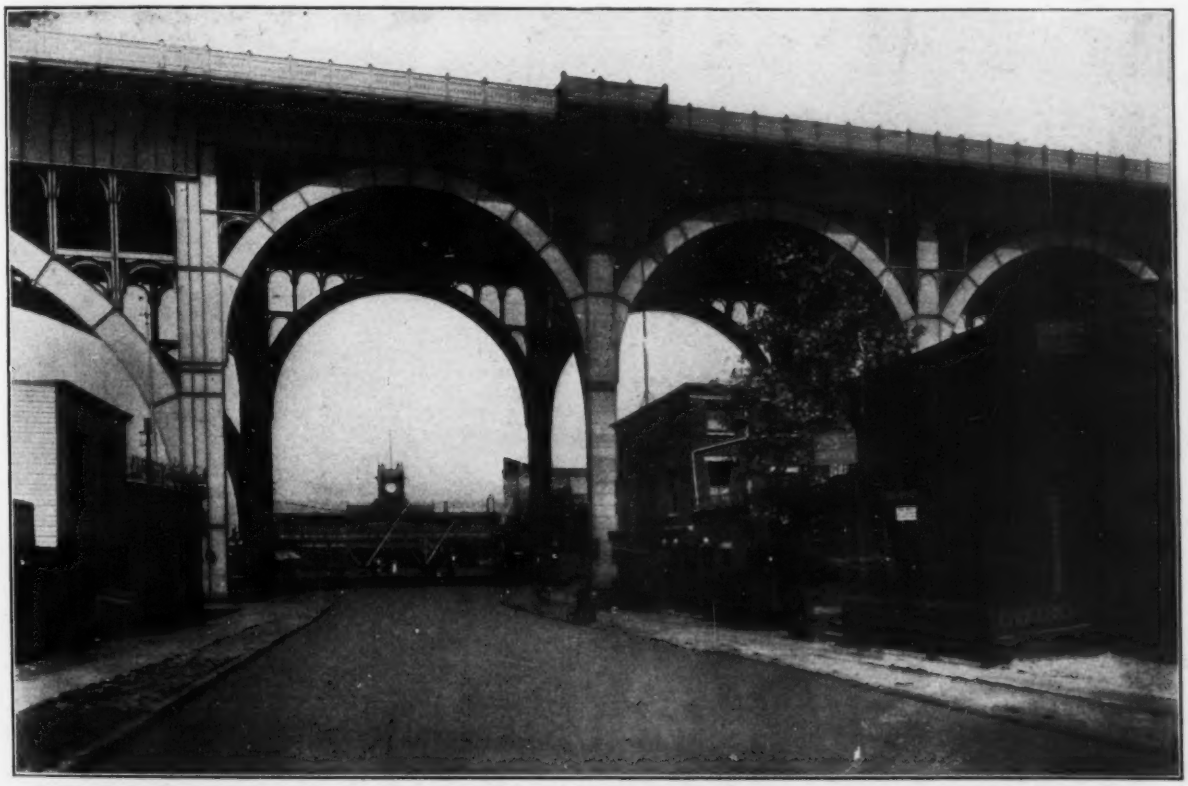
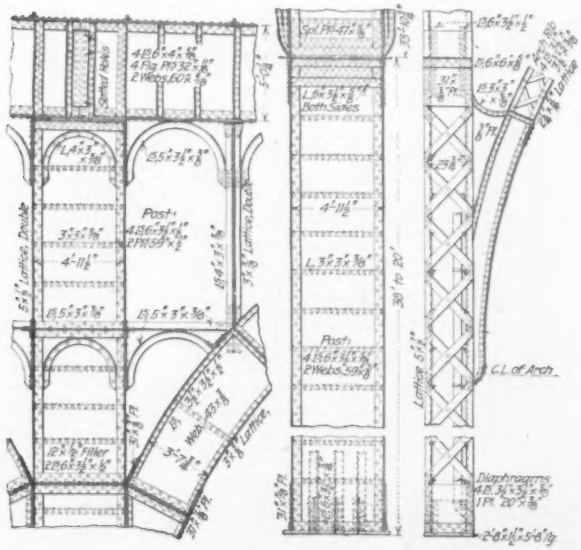
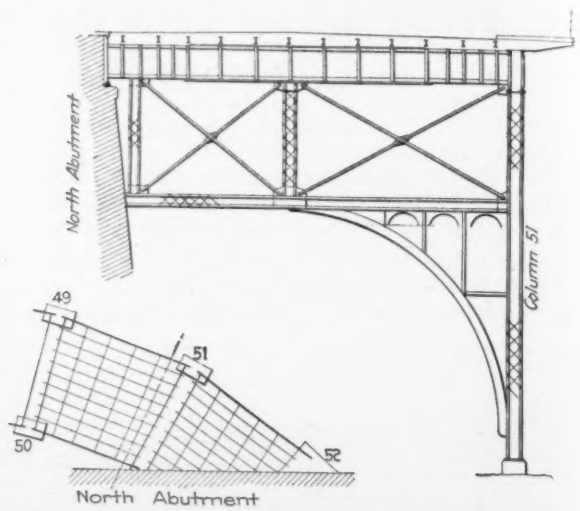


FIG. 1. RIVERSIDE DRIVE VIADUCT, SHOWING TWO OF THE SETTLED BENTS AT THE RIGHT; THE LARGE ARCH IS TO THE LEFT

Some time in 1912 the roadway on the viaduct was seen to show a marked sag in the length of a few hundred feet north of the large arch. Levels were taken on the footings, and it was found that two or three bents had settled, to a maximum of about 7¼ in. Presumably the settlement had existed longer, but maintenance and inspection during the dozen preceding years had apparently not been close enough to give information on the settlement. Plans for repair were made in 1915, and the work was taken in hand during the past summer.





FIGS. 2 AND 3. DETAILS OF RIVERSIDE DRIVE VIADUCT AFFECTED BY FOUNDATION SETTLEMENT

The columns are founded on a concrete spread footing supported by piles cut off about 2 ft. below water level, or 10 to 13 ft. below street level. Each column has 16 piles, giving over 8 tons dead-load per pile.

Why six of the columns should settle and the others not cannot be explained, but the Bureau believes that softer subsoil in the region of settlement is responsible; the location is near what appears to be the deepest part of the geological valley.

In spite of the settlement the engineers of the Bureau decided to rely on the old foundations without reinforcement or other change. This decision determined the general nature of the repair job.

The decision was based on three sets of levels, run respectively in April, 1915, September, 1915, and February, 1916. These showed irregular but not consistent differences; while the later levels showed generally lower elevations, they also showed in a few cases what appeared to be a rising. The conclusion reached was that the differences in levels were to be attributed to indefiniteness of the benches, personal equation of the leveling party, etc., and could not be held to indicate continuation of settlement. It was judged by the engineers of the Bureau that the settled foundations had remained stationary during the ten months of observation and in consequence that it is safe to leave the structure on the old pile foundations, merely jacking up the superstructure to original level.

It can readily be understood that this decision will save a large present outlay. Estimates made for repair with reinforcement of the six settled foundations indicated a cost of $100,000; the method followed is being carried through for about $11,000.

The columns of the viaduct are designed to carry about 450 tons total load, but the actual dead-load on them is only about 200 tons. All parts of the jacking rig were figured for the 200-ton load. The columns are unusually large and of peculiar section: A two-rib section with

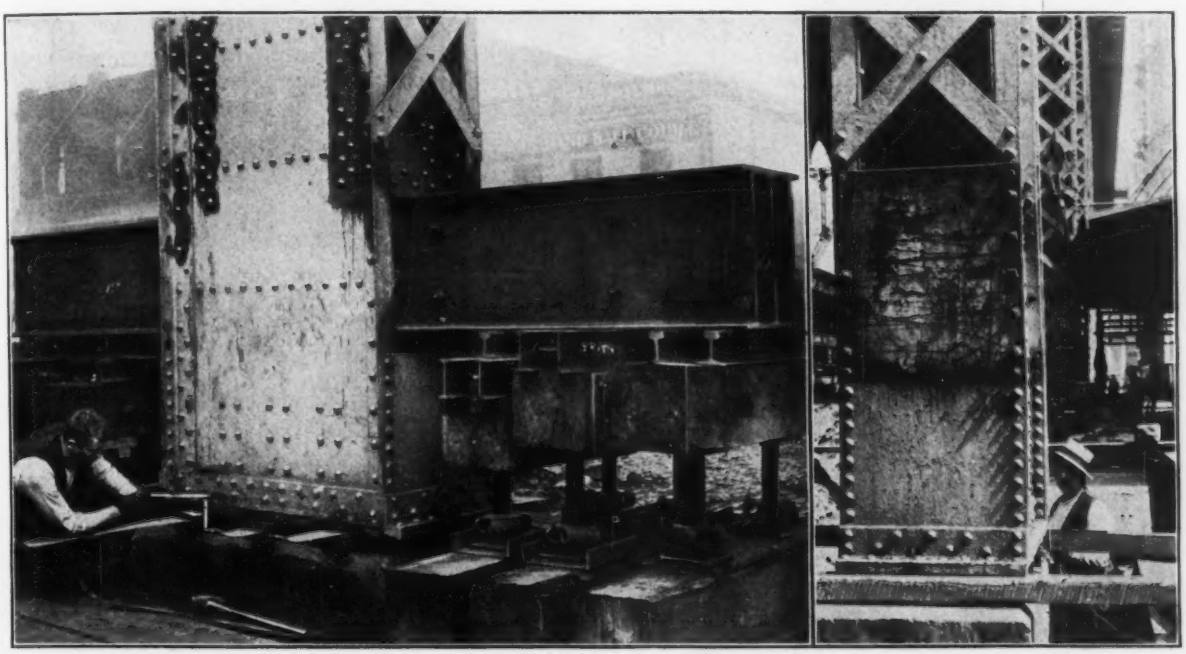
FIG. 4. DISTORTION AS SHOWN BY CHORD-LINE AND PLUMB-LINE (NORTH FACE OF COLUMN 15)

two 59x5/8-in. web plates, four corner angles 6x3½x5/8 in., and inner *horizontal* stiffening angles, the two channel-shaped ribs being latticed on the open (north and south) sides. Two jacking diaphragms were riveted vertically between the two webs, high enough above the base to allow the insertion of a jacking beam. The beam, consisting of three 24-in. I-beams, 16 ft. long, was placed through the latticed sides of the columns, bearing upward against the jacking diaphragms, and at the outer ends it rested on groups of jacks on cribbing.

This arrangement worked satisfactorily. The cribbing was proportioned for a soil loading of about 2 tons per sq.ft. and was placed in shallow pits about 4 ft. below street level. Screw jacks were used, of rated capacity under them, and the base was then grouted to make a solid mass with the cap stone.

After the first of the columns had been jacked and lowered, it was noticed that the entire base portion for a height of 8 or 10 ft. was buckled into a long S-shape (see Fig. 4). Such bending developed in several of the columns after the jacking, when the shims and wedges were placed under the base. The distorting process was facilitated by the absence of one panel of latticing, where the jacking beam went through the column.

Lateral displacement of the columns complicated the matter. All of the six columns were out of position before work started; three were east of true position by amounts of 1¼ in., 1¼ in. and 3 in. respectively.



FIGS. 5 AND 6. COLUMN 18 DURING AND AFTER JACKING, SHOWING REINFORCING PLATE AND RESIDUAL KINK

50 tons each. At first four jacks were used under each end of the triple jacking beam, but later six were placed at each end. The jacking was done on two jacks of one end at a time, four men working with an 8-ft. bar. It is believed that a lifting force of possibly 75 tons per pair of jacks was exerted.

Cast-steel jacks of the type ordinarily used in house raising failed by the threads shearing out of the nuts. Jacks of forged steel were then employed, with full satisfaction, except that the work was hard and slow. The men made a quarter-turn on one pair of jacks, then proceeded to the next pair and then to the third. It was thought during the work that hydraulic jacks would have proved more convenient, and amply safe if used with screw jacks as follow-ups. The amounts the six settled columns were jacked are shown in the accompanying tabulation.

West Side		East Side	
Col. No.	Raised, Ft.	Col. No.	Raised, Ft.
15	0.094	16	0.584
17	0.418	18	0.598
19	0.386	20	0.466

When the columns were jacked to the desired height or a little above, shims of I-beams or rails were set

This condition is attributed to either tipping or lateral shift of the top of the pile group under the column footing, occurring simultaneously with the settlement.

During the jacking the columns moved laterally back and forth, because of slightly unsymmetrical position of

FIG. 7. CRACKED AND SPALLED PIER CAPS ATTRIBUTED TO PRESSURE AND BONFIRES

the jacks. This kick at times amounted to several inches. In some cases its direction was such as to correct the initial displacement of the column; one column, however, is farther off true position now, after completion of the work, than originally. For the three columns whose

initial displacement was measured the figures are: Col. 18, before jacking, 3 in. E, after jacking, 2 in. E; Col. 19, before jacking, 1.2 in. E, after jacking, 0.6 in. W; Col. 20, before jacking, 1.2 in. E, after jacking, 3.4 in. E.

For one of the worst cases, Col. 15, a ram was used to bring the column back to true position. When the column was lifted, a plumb line from its top showed the bottom to be 1⅛ in. east, and the cap stone 2⅛ in. farther east; the column had originally been central on the stone. By using a battering ram the column was easily pushed over to plumb, and then the stone was shoved west 3¼ in. to come under the column base. At the two other columns where the ram was tried, it proved ineffectual. Sideward jacking by a kicking jack was resorted to. One of the columns was forced west ½ in. by this means, but the other one could not be moved over.

This latter, Col. 18, shown in Figs. 5 and 6, was seen to have a sharp kink in the base section even before jacking was started; this kink resulted from removal of two lattice bars to admit the jacking diaphragms. An extra tie-plate was put on the sides just above the batten plate at the base (Fig. 6). The kink partly remains, but it is less severe than it was during the work.

Dishing of the Column Webs

Fully as serious as the lateral bending of the columns was the dishing of the column webs, which appeared at the same time. The amount of this dishing on the 59-in. width of the rib was as great as ¼ in. in places. The longitudinal bending, measured at the corner angles, was

FIG. 8. NORTH END OF VIADUCT; COLUMN 51 IN FOREGROUND

at the same time as much as ½ in.; thus there was at the worst place a total buckling distortion of the web of over ¾ in. out of its plane, in a relatively short wave. This evidently denoted a very threatening failure of the web plates to do the work of load-carrying that they were expected to do.

These columns were not highly stressed, but on the contrary, with 100 sq.in. to carry the load of 200 tons, there was not more than 4,000 lb. per sq.in. in the metal if uniformly distributed. But it was quite evident during the work, that the column bases did not bear uniformly, and therefore probably put non-uniform stress into the columns. Some column bases were clear of the

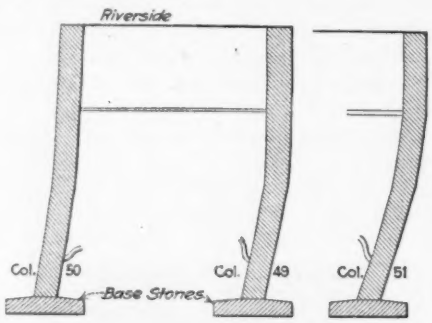


FIG. 9. DIAGRAM SHOWING RESIDUAL BEND IN VIADUCT COLUMNS
Horizontal scale 1 in. = 20 in.

footing cap stones at one side of corner; others had cracked or spalled the cap stones near one end. Thus the stress in the columns is a doubtful matter. In spite of this, the buckling and dishing of the columns is obviously a symptom of constitutional weakness.

Effect of Settlement and Jacking on the Viaduct

The settlement involved only six columns; if other parts of the viaduct settled, the movement was fairly uniform and its amount cannot now be determined.

The marked sag of the structure at the six settled columns did not produce any visible effects in the superstructure, except to shear off numerous bolts connecting the arches to the girders, due to longitudinal motion of the arches relative to the girders above them. During the jacking such longitudinal motion occurred in even more marked degree, resulting in shearing off most of the bolts in the connection.

Each longitudinal arch is connected to the girder over it by eight bolts at the crown. The minor (decorative) arches that fill in the spandrels are similarly connected to the girder by two bolts each. In some instances all eight bolts of the main arch and several of the minor-arch bolts sheared off during the jacking. These same bolts had been replaced only a year earlier, having then sheared off as the effect of the settlement. No rivets were sheared or heads snapped off in the attachment of arches to columns, so far as has been discovered.

Treatment of Column Bases and Footing Caps

The footing cap stones under the jacked columns will remain, except that in two cases the stone was badly cracked and was therefore removed and replaced by concrete. The other caps will, however, be banded with two 6x½-in. steel bands to forestall any cracking tendencies. Later, this banding will be extended to all piers of the viaduct.

Vertical stiffener angles 6½ ft. long, three on each web, have been attached to the sides of the jacked columns, to take up the dishing and guard against its recurrence. The base portions of these columns are being inclosed in a cast-iron shell 5-ft. high above street, fitting with a slight clearance around column and footing cap. Concrete filling will be carried up to the tops of these shells.

It is intended to apply this shell treatment to all the columns of the viaduct. Concrete filling 3 ft. high is being put in now. It is found that the base portions are often used to store paper and rubbish, which if set afire exposes the steel of the columns to possibly dangerous heating. Later the concrete filling will be carried up to near the tops of the columns, as fireproofing and as stiffening, in view of the uncertain stress distribution in the columns. The transverse portal or bracing arch is riveted to the side of the column, but is not backed by a diaphragm to transmit the arch thrust to both ribs of the column. Since the arch must exert thrust whenever it performs any bracing action or other useful effect, the need of some stiffening to the column at this point is obvious.

One Column Is Bent Five Inches Out of Line

Supplementary to the present repair work is some work on the northerly two bents of the viaduct, Cols. 49 and 50 and Col. 51. The latter is of serious concern in the viaduct.

Column 51 supports the west edge of the viaduct where it swings eastward to its landing on the north abutment (the abutment runs parallel to the center line of the viaduct and is situated just east of its easterly edge). The transverse bracing arch and strut from this column are attached at their east ends to the abutment; and as the abutment has moved outward several inches at the top, due to settlement of the forward edge, it forced the column outward near mid-height, forming what appears to be a permanent bend.

When this condition was discovered, the anchorage of the strut in the abutment was cut. This resulted in decreasing the bend in the column by ¾ in.; the strut moved east by this amount, and its east end also moved north 2 in. The residual bend in the column, which is clearly visible, measures about 5 in. (Fig. 8). Steel bolsters will be inserted through the pier cap stone to support the column, and the column will be filled with concrete.

There is a similar bend in both columns of the pair next south, Cols. 49 and 50. Their condition is sketched in Fig. 9. The treatment here will be the same as for Col. 51. The distortion of this bent is probably due to the action that pushed Col. 51 over, the effect being transmitted by the rigidity of the buckle-plate floor of the viaduct.

The pier cap stones under many of the columns are cracked, and some are spalled (Fig. 7). Bonfires may have contributed to this, but in some cases at least concentrated pressure of the column base is believed to be one of the causes. Ice formation in the bolt holes may also have played a part; the holes in the stones are 3 in.; while the bolts are 1¼ in.

It is held that the cracks are not important enough to justify replacement of the bases. As soon as an appropriation can be obtained, however, the stones will all be banded, and the cast-iron jackets placed.

Streets with Two Driveways and a Central Foot-Walk serve a part of the shop and warehouse district of Toronto, Ont. There being only a curb between the driveway and the front wall of the buildings, wagons can be driven or backed close up to the doorways. The driveways drain to and down their center lines. Midway in each block a cross driveway intersects the central walk. Poles and wires are placed at the curbs of the central walk.

What Is the Road to Progress in Engineering Education?

By Charles A. Holden*

The discussion now in progress in regard to engineering education in large part neglects two fundamental considerations: (a) A recognition of the basic principles common to all engineering problems; (b) the product being turned out—well-grounded men who will continue to be students.

Highway engineering may be taken as an illustration. It involves the following subjects that are common to most engineering work:

Mathematics
Physics
Chemistry
Graphics
Surveying
Earthwork
Drainage
Foundations
Retaining walls
Reinforced concrete
Hydraulics
Bridges
Electricity
Accounting
Specifications and contracts
Correct, concise and forceful presentation
Selection and management of labor
Administration, etc.

Also for a general and preliminary grasp of the subject, it is necessary to consider it from the viewpoint of one of the texts on highway construction. Later, if the engineer becomes enough concerned, intensive specialization is necessary.

In the same way other divisions of engineering activity can be analyzed into a group of subjects that are common and fundamental to all operations. Any one of these subjects may become the major activity in an engineer's life, while the others are minor but often-considered branches. One cannot be termed an *Engineer* who has not the background furnished by the above broad survey. Otherwise he must be denoted a [particular branch] engineer.

The consideration of an educational program always brings one to the inquiry whether the student is to be prepared for his lifework or for one branch of engineering. In the former case he needs a training that will enable him to accept any openings which may be offered him and to adapt himself to his environment. In the latter case there will be many prepared in a particular mold from whom the employer may select a few, while the others will have to enter different work for which they are not prepared and have to suffer the loss incident to the readjustment.

It is frequently two or three years before the young engineer finds a branch of engineering that combines opportunity and congeniality. It is then that with most effectiveness and certainty he can undertake specialization, either in actual practice or at some university.

The futility of too early specialization is illustrated from another angle by the reply to a circular in regard to training highway engineers. The commissioner of a state highway department writes: "I presume there is a lack of trained road engineers for the reason that the type of road building changes so rapidly that it is hard for an engineer to keep up with the times. I would give preference to graduates of a college highway-engineering

*Professor of Civil Engineering, Thayer School, Hanover, N. H.

course, provided they had as much experience as the other applicants."

The present time demands and the future will more insistently demand men with breadth and depth of vision, who will be able to discern fundamental principles, develop constructive policies and administer those policies consistently and adequately. I know of no one who is in a better position than the broadly trained engineer to render to this country and this generation the service that is needed.

Alignment and Drainage of Rural Highways

By H. E. Bilger*

SYNOPSIS—Description of small details essential to successful highway construction, but frequently neglected. Simple means of rendering grade crossings less dangerous, and the essentials of alignment to insure surface drainage.

The really permanent part of the entire improvement of a rural highway is its alignment, both horizontal and vertical. Hence the alignment merits attention before greater expenditures are made for surfacing work, which at best is of but a relatively temporary character. This statement applies particularly to the level prairie states of the Middle West, where to superficial appearances there seems little choice in the matter of alignment.

In the prairie states the general features of highway alignment as originally established, in most cases, will meet satisfactorily the requirements of modern traffic. These highways as a rule follow the section lines east and west, and north and south, and are straight.

Because of the unusually favorable conditions afforded by the topography and because also of the fact that practically all parts of the country could be utilized for agricultural purposes, a very large mileage of highways was laid out in the states of the Mississippi Valley.

The State of Illinois, for example, has 1.69 mi. of rural roads for each square mile of land area. Likewise Richland County, Illinois, has a corresponding figure of 2.27, the greatest in the state; while Pope County has 0.95 mi., the least in the state.

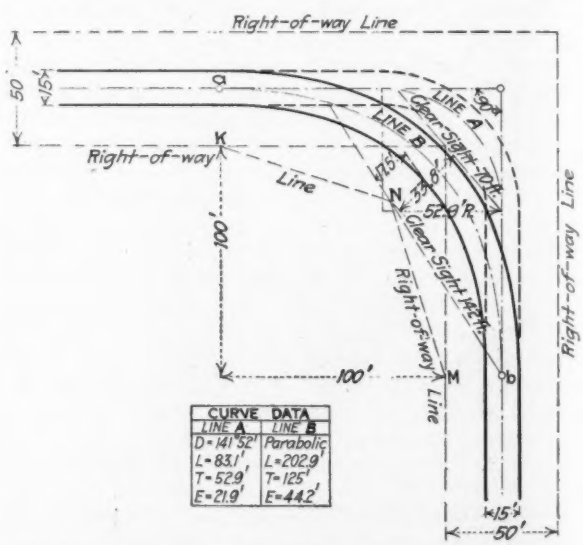
In view of this large mileage of intersecting roads on grades that invite high speed and on a soil that yields its best crops with the worst mud, it is evident that close attention should be paid to *all* details of highway alignment and particularly to those involving drainage.

Small Details of Horizontal Alignment Often Neglected

Marked improvement can be effected in small details that will tend greatly toward the general safety and comfort of the traveling public. For example, at all abrupt angles in the right-of-way, and particularly those approaching 90°, greater safety of travel can be brought about by simply procuring enough additional right-of-way to permit moving the inner fence corner a distance of only 30 or 40 ft. By so doing, the radius of the curve can be considerably lengthened and a much better view provided. Travel is benefited out of all proportion to the cost of the little piece of land. In order that this additional right-of-way at the corner may be utilized to the utmost, it is necessary that all vegetation be kept down and that signboards, if any, be transferred elsewhere.

A typical 90° angle in a highway is shown in the accompanying sketch, similar to which there must be more than 100,000 in Illinois alone. To keep the improvement at the corner within the 50-ft. right-of-way lines would require a curve having only a 52.9-ft. radius, and the clear sight provided for the traveler would be only 70 ft.

As contrasted with this, notice the advantages afforded by acquiring the piece of land *KLMN* (see sketch), which contains 0.055 acres and would cost, at the rate of $300 per acre, only $16.50. The longer curve provides an outlook of 142 ft. as against the 70 ft. for the other. At the same time it shortens the pavement from *a* to *b* by an amount sufficient to save 40.6 sq.yd. for a 15-ft. pavement—about $70 on a brick pavement and $50 on one of concrete, which is about three times a reasonable price for the land.



RECOMMENDED CURVE FOR 90° TURNS; ILLINOIS HIGHWAYS

In addition to this the right-of-way is not contracted at the corner as it is by the shorter-radius curve, so that the 17.5-ft. margin between the inner edge of the pavement and the property line is preserved for use as an earth road around the corner as well as on tangents. Moreover, the land *KLMN* because of its corner location, is practically valueless for agricultural purposes.

At all crossroads additional land should be provided by moving the fences back to include the land at the four

*Road Engineer, Illinois State Highway Department, Springfield, Ill.

corners within the limits of the right-of-way, as proposed in the sketch. This brings the control of the corners within the authority of the road officials, who can have removed all vegetation that obstructs the view of the approaches.

Where there is an angle greater than 45° in the alignment and the improvement at this place is on an embankment higher than, say, 4 ft., there should be provided on the outside of the curve a substantial-looking guardrail painted white, to serve as a warning to traffic.

Improvements at Railway Grade Crossings

In the State of Illinois there are 95,696 mi. of rural highways and 13,198 mi. of steam railroads. As practically all the highways have been located on the section lines, their directions are north and south, and east and west. With comparatively few exceptions the general directions of the railways are also north and south, and east and west, so it becomes at once apparent what a relatively large number of crossings are formed by the railroads and the highways; and considering the practically level topography it is readily seen why 87%, or 17,119, of these crossings are at grade.

Economic conditions do not exist that will at this time warrant the separation of grades on as many as 10% of these crossings in the rural parts of Illinois. Nevertheless, every effort should be made to improve these crossings in a manner that will minimize the accident hazard.

In the vicinity of crossings the improvement should be such as to provide an unobstructed view along the track for a distance of at least 1,000 ft. each way, and this view should be enjoyed along a length of highway extending a distance of at least 300 ft. on each side of the crossing.

It is particularly important that there be provided accommodations for two vehicles to pass with entire safety and comfort at the crossing, requiring never less than 18 ft.; and the highway for a distance of 50 ft. or more on each side of the crossing should be practically level or should at least have a very easy grade, say not steeper than about 2%.

Much destruction of both life and property is caused by motor vehicles taking a run on the steep grades up to the crossing and then stalling the engine on the track. Occasionally two vehicles will make a run from opposite sides of the crossing and upon reaching it find that the highway does not have sufficient width for passing.

Vertical Alignment and Drainage

In establishing the general vertical alignment of the highway there must be carefully considered the construction cost, the serviceability of the road and the practicability of economic maintenance. These three factors will assume varying degrees of importance, dependent largely upon the development of the country contiguous to the road improvement.

The proper alignment for any road is so closely related to such local conditions as topographic features, available funds, etc., that general recommendations might be wholly a misfit when applied to a particular case. It can be safely said, however, that grades should be kept reasonably consistent and closely in harmony with the general topography of the neighboring country, but aside from this there is little justification for expenses incurred in order to obtain grades lower than some 3 or 4%.

To design highway improvements according to a limiting grade fixed for a geographical area even as large as 15,000 sq.mi. is not warranted either by the nature of or by any factors of the highway problem. The whole matter is more of a local issue as regards both the factors of construction and the use of the roads, but in order to make the most out of a given combination of conditions a broad experience in road building is required.

The Most Essential Element in Surface Drainage

The particular feature of a large percentage of the Illinois roadwork commanding attention is the necessity for positive surface drainage of the roadbed by elevating it above the level of the adjacent fields. In order to maintain satisfactorily a road surface in practically level country on other than a sandy soil it is generally absolutely necessary to forestall capillarity by elevating the roadbed.

It is probably true that on more than one-half of the mileage of the roads in the state the traveled way is lower than the general elevation of the adjacent fields, and in a large percentage of these cases the practicable gradient of the side ditches is not sufficient to carry off surface water. This condition is due to the simple fact that formerly highway construction consisted chiefly of fencing off a right-of-way across the prairie without attention to the matter of surface drainage.

In connection with tilling the land for agricultural purposes the under-drainage of the road has been given more consideration than the surface drainage. It has not been fully appreciated that underground drains are primarily for the removal of underground water and that for the removal of surface water these drains cannot be depended upon as more than a 20% substitute for complete surface drainage.

Classification of Soils in Facility of Drainage

With respect to their properties of shedding or absorbing surface water all characters of soil encountered in Illinois roadwork may be assigned to one of the three following classes:

Class 1—Impervious soils, such as the prevailing gumbo throughout the corn belt, or any dense clay or other soil through which surface water will not readily penetrate.

Class 2—Semi-pervious soils, those soils of an intermediate character that evidently are not assignable to either Class 1 or Class 3.

Class 3—Very porous soils, including sand, gravel or loose stone; soils through which surface water will readily penetrate.

In case of doubt as to whether a particular soil should be assigned to Class 1 or Class 2 it should be assigned to the former, and for doubt between Classes 2 and 3 the assignment should be made to Class 2.

How To Deal with Each Class of Soil

Class 1—No matter whether on an elevated tableland or on a low bottom, where the obtainable side-ditch grade is less than 0.4%, not more than 800 ft. of road in one stretch should have the crown of its subgrade (finished crown for earth roads) less than 12 in. above the general elevation of the adjacent land without the limits of the right-of-way, notwithstanding the size of the side ditches.

Exceptions to this rule would be where the highway follows a practically level ridge and the adjacent land drains away from the road on both sides; also, where the highway follows along land that all drains one way. In the latter case, needless to say, cross-culverts should be placed at

suitable intervals to carry the water from the higher ditch to natural outlets.

Class 2—Same as Class 1, except that 0.2% should be substituted for 0.4%.

Class 3—The crown of the subgrade should be at least 6 in. above the general elevation of the adjacent land without the limits of the right-of-way.

Notwithstanding the character of soil, where the grade of the side ditches is from 0.4% to 1% an endeavor should be made to keep the crown of the subgrade at least 6 in. above the general elevation of the adjacent fields. With a nonporous soil and an open-ditch gradient of less than 0.4% it is difficult to provide surface road drainage by seeking natural channels located at considerable distances apart. A more effective and more positive practice is to elevate the roadbed by filling from borrow pits to an elevation above the accumulated water level, which under the conditions assumed is the general level of the adjacent fields.

Reduce Speeding Hazards on Vertical Curves

The speed of motor-vehicle traffic requires that other details of vertical alignment be given full consideration. For the most rigid types of Illinois roads, as brick and concrete, parabolic vertical curves should always be used when the rate of change from one grade to another is as much as 1%, and for the more resilient types of roads when the rate of change is as much as 2%. Both the safety and the comfort of travel require these vertical curves, though their proper lengths are dependent upon factors that necessarily must be assumed.

It would be only reasonable to assume that in rural territory automobiles frequently ascend short grades up to 10% at a rate of from 15 to 25 mi. per hr. To come from this speed to a full stop, there should be provided at least 150 ft. If it is assumed that on ascending grades two automobiles are approaching one another, each at the rate of, say, 20 mi. per hr., the vertical curve at the summit should be of such length as to enable the drivers to see one another when they are at least 300 ft. apart.

With the eyes of the drivers 5 ft. above the road surface no vertical curve is required to meet this condition when the change in the rates of the grades is not more than 6.67%. When this change is 10%, however, the curve should never be less than 200 ft. long. Likewise, changes of 13 and 16% would require minimum curves of 292 ft. and 360 ft. respectively. Practical considerations would commonly suggest the use of vertical curves before they would be required to meet this condition, but nevertheless it is well to have in mind reasonable limits within which ordinary practice should be confined.

While the search for the ideal road-surfacing material and the battle for supremacy of certain material interests go on, the local road-building authorities cannot do better than confine their activities to the fundamentals of the highway problem—namely, the establishment of a drainage practice similar to that of the railways and the production of a highway alignment that will properly fit *all* the purposes for which it exists.

Plans for a Municipal Electric Light Plant for St. Louis, Mo., have been under consideration for 18 months. By a resolution of the Board of Aldermen on Oct. 19, Mr. Hooke, the Director of Public Utilities, was authorized to make additional surveys and estimates for this plant at a cost of $12,500, and to report within a year's time.

Proposed Code for the Sanitary Control of Waterways*

I. There shall be no objectionable deposits at any point in a waterway as a result of sewage or other wastes discharged therein.

Note—To prevent such deposits public authorities should recognize the efficacy of such preventive and corrective measures as the dredging and removing of the deposits before they become objectionable, the proper location of sewer outlets and the removal of solids by screening, sedimentation, chemical precipitation and like methods.

II. There shall be no local nuisance created at or in the vicinity of any sewage or industrial-wastes outfall, arising from excessive turbidity or the production of odors.

Note—To prevent such conditions public authorities should recognize the efficacy of such preventive and corrective measures as the arrangement and proper protection of sewage outlets and all methods of sewage treatment in properly designed plants, including screening, sedimentation, chemical precipitation, oxidation in various types of coarse- and fine-grained filters and treatment by the activated-sludge process.

III. There shall be no general nuisance created in a waterway due to excessive turbidity or to odors, as a result of sewage or other wastes discharged therein.

Note—To prevent such conditions public authorities should recognize the efficacy of such methods of sewage treatment as are enumerated in the note to Article II, but it will be necessary to carry the treatment far enough to meet the following requirements:

1. That dissolved oxygen shall be present in the waters of the waterway at all points and at all times in such quantities as to prevent the emanation of putrefactive odors.

2. That every municipality or industrial establishment discharging sewage or wastes into a waterway may have its proper share of the dissolved oxygen, subject to the aforesaid limitation of paragraph 1.

3. That the share of dissolved oxygen of a waterway, which a municipality or industrial establishment may utilize at any time, should be subject only to the aforesaid limitation (stated in paragraph 1), so long as other parties are not utilizing it nor thereby being deprived of its use.

4. That as the size and number of municipalities or industrial establishments discharging sewage or wastes into the waterway increases, and the available share of dissolved oxygen for each decreases, treatment methods must be increased generally and correspondingly, in order that each party may continue to have the share of the available oxygen to which it is entitled.

IV. There shall be no interference with or undue burden upon mechanical operation or bacterial efficiency of water-purification plants procuring their raw water from a waterway.

Note—The public authorities should recognize in this case:

1. The efficacy of relocating water-supply intakes and sewer outlets as a means of remedy.

2. The efficacy of such methods of sewage treatment as will reduce the quantity or alter the character of either that suspended matter or that dissolved matter, or both, which may interfere directly or indirectly with the mechanical efficiency and operation of water-purification plants.

3. The efficacy of such methods of treatment as will reduce the number of bacteria that may cause a bacterial burden upon water-purification plants.

4. That since water- or sewage-purification processes can be carried to any desired limit, at least theoretically, the share of responsibility that is to be imposed upon the different sewage-disposal and water-purification plants, with respect to extent of purification, will obviously depend upon a number of legal and administrative considerations of an

*From the report of the Committee on Sanitary Control of Waterways, submitted to the Sanitary Section of the American Public Health Association at Cincinnati, Ohio, Oct. 26, 1916. The members of the committee are Harrison P. Eddy, chairman, Boston, Mass.; Edwin A. Fisher, Rochester, N. Y.; Paul Hansen, Springfield, Ill; Theodore Horton, Albany, N. Y.; W. H. Dittoe, Columbus, Ohio. The code was preceded by a statement of fundamental principles. The full report will be printed in the "Journal" of the American Public Health Association (377 Boylston St., Boston, Mass.). Reprints will be sent to members of the Sanitary Section about 30 days in advance of the 1917 convention, at which the code will be presented for possible amendment and for adoption.

economic, engineering and public-health nature which properly come within the jurisdiction of central authorities and the courts.

V. There shall be no active (potentially dangerous) bacterial contamination or gross pollution of properly located and authorized shellfish beds.

Note—Public authorities should recognize in this matter five important features:

1. The proper supervision, by competent authority, of the location of shellfish beds and drinking places, and suitable control of shellfish operation.

2. That the gross pollution of shellfish beds by deposits of suspended matter from sewage or industrial wastes can be prevented, and that bacterial contamination of the beds may be avoided, by relocation of sewer outlets.

3. That the gross pollution of shellfish beds by deposits of suspended matter from sewage or industrial wastes can be prevented also by the installation of proper sewage-treatment processes that will remove these solids to the extent necessary.

4. That where shellfish beds are so located as not to be subject to gross pollution and where bacterial protection only will be necessary, this protection can be accomplished satisfactorily by the employment of such methods of treatment as will destroy or remove the bacteria.

5. That since active bacterial contamination and gross pollution of shellfish beds can be prevented by proper location of such beds and by proper location of sewer outfalls and proper treatment of sewage and other wastes, the enforcement of the abandonment or relocation of shellfish beds or of preventive or remedial measures to protect such beds will obviously depend upon a number of legal and administrative considerations of an economic, engineering and public-health nature which properly come within the jurisdiction of central authorities and the courts.

VI. There shall be no active (potentially dangerous) bacterial contamination or gross pollution of properly located and authorized bathing beaches and other bathing places.

Note—The public authorities should recognize in this matter:

1. That notwithstanding the legal rights of riparian owners, in view of our knowledge of the dangers to health arising from bathing in waters contaminated by the bathers themselves as well as from sewage and wastes discharged into them, control is needed both in the selection and authorization of bathing places and of the discharge of sewage and wastes into the waters in which bathing is authorized.

2. That the gross pollution of bathing beaches by suspended matter from sewage or industrial wastes can be prevented, and bacterial contamination may be avoided, by relocation of sewer outlets.

3. That where protection of properly authorized bathing places, from bacterial contamination only, is required, such protection can be secured by such sewage-treatment processes as will destroy or remove the bacteria.

4. That where properly authorized bathing places require protection against gross pollution arising from suspended or other objectionable sewage matters, this additional protection can be secured by the installation of such additional sewage-treatment processes as will sufficiently remove the objectionable matters.

5. That since active bacterial contamination and gross pollution of bathing places can be prevented by proper location of such places and by proper location of sewer outfalls and proper treatment of sewage and other wastes, the enforcement of the abandonment or relocation of bathing beaches or of preventive or remedial measures to protect such beaches will obviously depend upon a number of legal and administrative considerations of an economic, engineering and public-health nature which properly come within the jurisdiction of central authorities and the courts.

Position-Light Signals on the Pennsylvania R.R.

Signals that use lights only (by day and night), but in which the lights give position indications simulating the positions of semaphore arms, have been in use for about 15 months on the electrically operated section of the Pennsylvania R.R. between Philadelphia (Broad St. station) and Paoli, Penn.—20 mi. All lights are white, since all indications are given by position (Fig. 1).

Each unit or signal consists of a group of incandescent lamps inclosed in casings and mounted upon a board or background. Certain of those lamps are lighted or extinguished in varying the indications. Fig. 2 shows one of the installations on a four-track signal bridge near Ardmore. In this view the eastbound train has just set the top right-hand signal at "Stop," while the signal below shows that there is a train in the second block ahead. In the left-hand signal the top indication is "Proceed, prepared to stop at next signal"; the lower indication shows that there is a train on this track also in the second block ahead. The upper signal (for the first block) gives three indications; the lower signal (for the second block) gives two indications. The blocks here are 3,500 ft. long. The view shows a steam train, but the catenary trolley-wire suspension will be noticed.

Exact and permanent location of the lamps behind their respective lenses is essential for correct reading by the motormen. For this reason bayonet sockets are used in preference to screw sockets. Difficulty was introduced in the beginning by the fact that the lamp makers could not guarantee a uniform distance between the base of the lamp

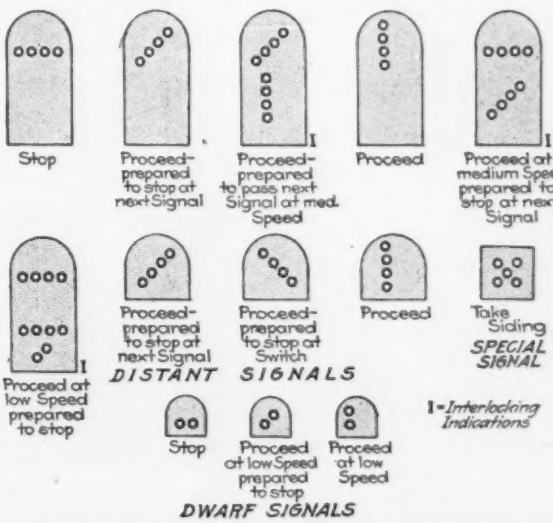


FIG. 1. TYPICAL INDICATIONS BY POSITION-LIGHT SIGNALS ON THE PENNSYLVANIA R.R.
Those marked "I" are for interlocking plants

and the filament, and allowance had to be made for a variation of at least ¼ in. A slotted brass collar is used for adjusting the lamp in position.

Although the filament has coils of large diameter, there was trouble due to its attachments not being secure and permitting a drop, or sag, that necessitated refocusing. This defect has been remedied. The lamp has a spherical bulb, and the filament is above the center line.

The first signal was mounted about 25 ft. above the rail, but at a distance of 175 ft. the light was invisible. A special lens was then devised, in which both the lower quarter and the bullseye portion deflect rays downward. This was necessary as the signal bridges are about 29 ft. clear above the rails, and the top light of the vertical row of lamps is over 40 ft. from the rails. The effect is to give practically three zones of light: First, the direct rays, from a distance of a mile or more to within 200 or 300 ft. of the signal; then, overlapping these and to within 40 or 50 ft. of the signal, come the rays deflected downward through the lower quarter of the lens; finally, the

image in the mirror reflected through the bullseye. The steps of the lens are painted black to prevent trouble from the reflection of rays of sunlight striking these steps. On curves the same type of lens is used, with the spread-light feature in addition.

The signals are operated with 3-volt current by day and 11-volt current at night. The latter gives good results on foggy nights. It is found that a 12-volt lamp burning night and day at 10 volts has a range of 3,000 ft. or more by day and is not hard on the eyes at night if the signals are a mile or two apart. The first signals had (for each indication) five lights in a row, 12 in. c. to c., corresponding to a 5-ft. semaphore arm. Better results are obtained with four lights 18 in. c. to c., as in Fig. 1. In this illustration the backgrounds appear unduly large, but in construction the second and third rows of lights have their own backgrounds, of various shapes, depending upon the number of indications. This reduces the wind pressure on the structure.

This system is in use for automatic and manual block signals, interlocking signals and dwarf signals. There is

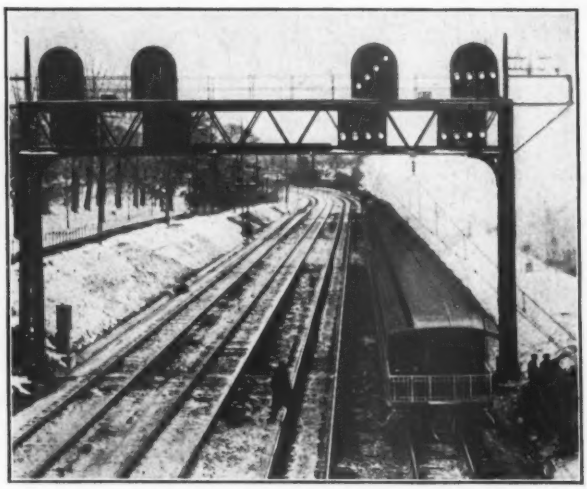


FIG. 2. SIGNAL BRIDGE WITH POSITION-LIGHT SIGNALS ON PENNSYLVANIA R.R. NEAR PHILADELPHIA, PENN.

also a special "Take siding" signal (Fig. 1). This is mounted by itself on a mast and has a background 3x3 ft., with lights 12 in. c. to c. It is lighted only when giving the indication. In December, 1915, two signals were installed to protect traffic at a signal-track tunnel. These are lighted by the approaching train.

Results with Light Signals

After a year's experience it is concluded that with the position-light signals the chance of false clear or misread signals is at a minimum, while the number of total failures is greatly reduced. The enginemen are practically unanimous in stating that these signals are more visible than any others at night, in fog and in snowstorms. The close indication (within 150 ft. of the signal) is better than that of the semaphore at night and is as good by day as the semaphore is at night.

The indications are uniform by day and night. With semaphores the indications are by position during the day and by color during the night, and by both during morning and evening periods. Colored-light signals give indications by color only, while position-light signals give indications by position only. Semaphore and colored-light signals are limited to three indications. The position-light signals can give four indications, thus in many cases eliminating advance or other signals required to supplement the semaphore.

These signals are said to have every advantage where power from a central plant is available, as where alternating-current track circuits are used. They are specially adapted for distant switch and block signals on branch lines. The cost of lighting continuously from a primary battery is prohibitive, but on lines of light traffic the signals can be lighted on the approach of a train and extinguished after the train has passed. The cost of a typical four-track automatic-signal bridge with signals and all fittings (including track circuits) is estimated at $10,071, as against $10,633 with motor-operated signals, which are about the least expensive semaphore signals on the line. With 1,882 lamps in service the lamp renewals average only 179 per month, or a cost of $16 per year, which is about the cost for ordinary repairs to motor-operated semaphore signals.

This system of signals was developed by the engineers of the Pennsylvania R.R. in coöperation with the Corning Glass Works and the Union Switch and Signal Co. As several parties had a share in the invention of various parts, it was considered advisable to pool the patents and applications. Licenses under these patents have been granted to the Union Switch and Signal Co., General Railway Signal Co., Federal Signal Co. and the Hall Switch and Signal Co. The foregoing description is compiled from an article prepared by A. H. Rudd, Signal Engineer of the Pennsylvania R.R., and sent out by the railway company. In it Mr. Rudd makes the following statement:

"My own opinion [of these signals], prejudiced perhaps but nevertheless concurred in by many of our people who have had to do with them, is that unless a speed control is developed, so reliable and dependable as to preclude the necessity of fixed signals, the power-operated signals of the future will be light signals (either position or color), perhaps of several types, superseding gradually the movable arms now in use."

Photogrammetry for Taking Topography of Watershed

By Douglas H. Nelles*

Photogrammetry, on this continent at least, has been made use of principally by governments where large areas were to be mapped in rough, rugged, mountainous regions. In Canada 22,750 sq.mi. has been mapped by this method; and if to this is added the 31,000 sq.mi. mapped by the Canadian International Boundary Commission in Alaska, it makes the total of approximately 53,750 sq.mi. to date.

The recent use of photogrammetry for mapping 163 sq.mi. of the Thirty-one Mile Lake watershed, Quebec, was, however, of an entirely different nature. This presented a mass of irregular, bush-covered hills from 500 to 1,000 ft. high. Running through the center of the watershed are two main lakes, Thirty-one Mile and Pemichangan, and in between the hills on either side are 101 small lakes.

The Geodetic Survey of Canada had two primary triangulation stations established in this vicinity, one in

*Ottawa, Ont.

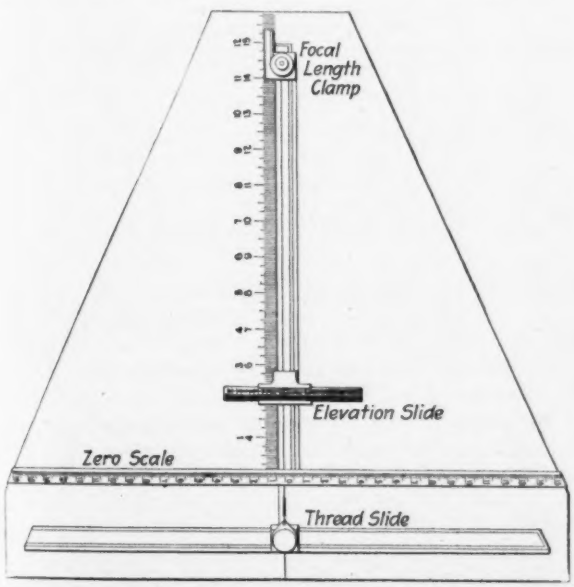
the center of the watershed and one on the outside of the southern edge, from which a secondary triangulation was thrown over the watershed, comprising 24 stations. Ten of these stations were in the southern portion of the watershed, containing 93 sq.mi.

To these 10 stations were tied 83 photo-topographic stations, and from each station was taken a set of photographs forming a panorama of the country that could be seen from each station. Plotting from photographs taken during the summer was not a success, as enough points could not be identified to control the topography. Another set of photographs was therefore taken during Feburary, from which very successful results were obtained.

All the lake outlines, creeks, farms and clearings were surveyed by plane table, and the roads by transit and stadia. All these detail surveys were tied to the triangulation and camera stations. The rest of the topography was filled in from the photographs.

Office Work and Methods

The office routine consisted of computing the positions of stations; drawing a polyconic projection on the scale chosen; plotting triangulation and camera stations; transferring and adjusting the plane-table topography to the map; drawing the horizon and direction lines upon the photographs; identifying and marking direc-



SIMPLE DEVICE FOR PLOTTING TOPOGRAPHY FROM PHOTOGRAPHS

tion points; choosing and marking topographic control points on photographs; plotting the photograph traces on the map, as explained later; plotting control points; determining their elevations and finally sketching in contours and other detail.

The ordinary methods of plotting from photographs are generally known and will not be described. In this work a special machine shown in the accompanying illustration was used entirely for taking out elevations. The credit for the original idea of determining elevations by the method to be described is due to W. H. Boyd, Chief Topographer of the Canadian Geological Survey. The machine designed by the writer and made in the Dominion Observatory machine shop, besides giving elevations, measures the focal length of a photograph and plots the photograph trace on the map.

The focal-length clamp has a double action. The larger screw clamps the scale in position. Inside of this screw is a smaller one with a fine steel point on its lower end. The smaller screw serves the purpose of lifting the needle point so that it will not touch the paper when the machine is being used for determining elevations. This point is inserted at the station point when the machine is being used to plot photograph traces or to determine focal lengths.

The focal-length clamp has a vernier with two zeros, one at the forward edge, from which the thread swings, and one opposite the center of the needle point. The vernier reads to 0.01 in. In the center of the forward edge of the focal-length clamp is a small hole in which a fine silk thread is fastened by a wooden plug. The other end of this thread is attached to a spring that serves the purpose of keeping it taut. The spring is attached to a ring placed around the screw of the thread slide. The thread-slide clamp is kept tight enough so that the slide will stay at any position.

Use of Plotting Machine

To determine an elevation: Set the focal-length clamp at the focal length of the photograph, using the line of the zero scale as zero (see illustration). Set the altitude scale of the elevation slide so that the part of it opposite the center mark of the slide will read the altitude of the station plus the height of the camera. Now measure the distance between the control point and the photograph trace and make the distance between the elevation slide (on the focal-length scale) and the zero scale equal to it. Measure the distance of the control point above or below the horizon line and set this distance off on the zero scale above or below the zero. Move the thread to cut the point so obtained, and the thread will also cut the altitude scale of the elevation slide at the elevation of the station.

When working on small scales and in a region of high altitudes such as the Rocky Mountains, points are very often plotted at a distance of from 5 to 15 mi. from the station. This would only be done when nearing the limits of the country controlled by the survey, and in these cases it becomes necessary to take into consideration the correction of curvature and refraction, which is 14.4 ft. for 5 mi., 57.4 for 10 mi. and 129.1 for 15 mi. This is done by computing and plotting a curvature and refraction scale for the particular scale of the map, and so placing it on the machine that the forward edge of the elevation slide will cut it at the proper point each time that it is set. Then to get the true elevation of the control point, the curvature and refraction reading is mentally added to the thread reading.

To determine the focal length of a photograph with the machine, a photograph is selected that has two well-defined points in it, to which instrumental directions have been read from the camera station. These two directions are plotted on the plan. The center mark of the forward edge (bottom in view) of the machine is then placed over the direction line of the photograph; and the distances of the points on each side of it are marked on the edge of the machine with a pencil. The larger the angle between the points the better the results.

The needle point of the focal-length clamp is screwed down at the station and the clamp left loose to slide longitudinally. The machine is then swung around and moved up and down about the clamped needle point until the two pencil points on the edge coincide with the two direction lines on the plan. The machine is then oriented in azimuth, and the reading on the vernier opposite the needle will give the focal length of the photograph to 0.01 in.

If a line is drawn along the edge, it will be the trace of the photograph. If a dot is made at the center mark on the edge and a line drawn between it and the station mark, it will represent the direction line of the photograph.

The photographs were generally enlargements, double the size of the original, and measuring 12 by 8½ in. They were all made the same size, so that their focal lengths were the same; but before being used for plotting, the focal length was checked by measuring the distance between the cone marks that control the location of the horizon and direction lines. All recent surveying cameras have marks placed at a distance apart equal to the focal length, which are photographed upon the negative so that the focal length of any photographic print or enlargement can be measured directly.

Photographic Surveying with an Ordinary Eastman Kodak

It so happened that a map being plotted under the direction of the writer was found to have what is officially termed "a hole in it"; that is to say, there was a small valley which could not be plotted for lack of photographs. It was known that the valley contained a small lake that had not been surveyed by plane table, so that it was necessary to get accurate information.

An officer of the survey could not be sent to the locality to get the required photographs, so it was decided to ask the superintendent of a fish and game club who lived near the valley to take the photographs for the survey. He understood how to use a kodak, but not a surveying camera, so a No. 4 Eastman kodak was sent to him with a map showing the hills surrounding the valley and the two places from which the photographs were to be taken, together with minute directions for exposure, etc.

In order to use these photographs it was necessary to locate from them the position on the map of the two places from which they were taken and the position of their traces on the map. To do this, a photograph trace was drawn upon a piece of tracing cloth. This was taken to represent the trace of the photograph on the right side of the panorama from one station. The trace of the photograph next to it was then plotted in its proper position by making use of a common object in the overlap of the photographs, and the other three traces plotted in the same way. There were eight known points shown in the different photographs which had been plotted on the map from other sources. Direction lines were drawn through these points on the tracing cloth, which was then placed on the map and the position of the station located by the three-point problem, or in this case an eight-point problem—that is, a position was found for the tracing cloth so that every direction line cut through its respective point on the map. When this position was found, the position of the station point was pricked through on the map and also positions of the photograph traces.

New Formula for the Crown Thickness of Arches

By Joseph P. Schwada*

The usual procedure in the design of arches requires the adoption of an arch ring to which the rigid analysis may be applied. If the arch ring chosen is satisfactory, as shown by the analysis, the work is over; but if it is not satisfactory, repeated trials must be made until a rib is found that satisfies the requirement for working stresses. If a certain variation of thickness from crown to springing is adopted, the problem becomes one of choosing a proper crown thickness.

This problem has been studied to some extent, and as a result a number of empirical rules have been formulated for its solution. Application of these rules shows large differences in results, and so will lead one to think that the assumptions upon which some of them are based are radically wrong or that they ought more properly to be called personal opinions.

The important factors entering into the problem are: The span and rise of the arch ring, the curvature, the thickness at the crown, the relative thickness from the crown to the springing, the live- and dead-loads, and the amount of temperature variation and unit stresses assumed in the design. The number and nature of these variables,

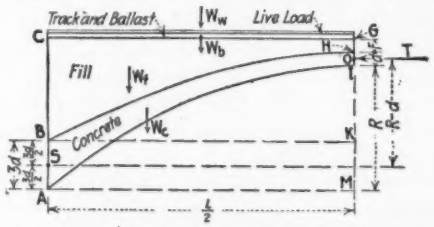


FIG. 1. NOMENCLATURE USED IN ARCH-THRUST FORMULA

however, are such as to prevent inclusion of all of them in the same formula, because no rule can be drawn up that will include such variables as curvature and temperature variation; some assumptions regarding them must therefore be made and a formula drawn up accordingly.

With this in view a method is here proposed for the determination of the crown thickness of arches. The assumption is made that the vertical thickness at the springing is equal to three times the crown thickness. In the design of a series of railroad arches analyzed according to methods followed in actual practice, it was found that, when this relation was adopted and the arch axis made to coincide with the dead-load pressure line, satisfactory results invariably were obtained. It is further assumed that the intrados and extrados curves are parabolas, because in the design of these arches it was found that, though arcs of circles were used, the parabola closely approximated the curves, sufficiently at least for the proposed method. A uniform live-load on the whole span is assumed, and a temperature variation of ± 20° F. is allowed.

With these assumptions an expression for the crown thrust can be formulated. The depth of arch ring at the crown can then be determined by dividing the thrust by an average unit crown stress, a stress which has some definite relation to the maximum stress in the arch ring

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for various spans and ratios of rise to span. In the sketch of a half-arch, Fig. 1, let

L = Span of intrados, ft.;
R = Rise of intrados, ft.;
d = Crown depth of ring, ft.;
F = Depth of fill at crown (not including track and ballast or pavement), ft.;
f = Weight of fill, lb. per cu.ft.;
B = Weight of track and ballast or pavement, lb. per sq.ft.;
c = Weight of concrete, lb. per cu.ft.;
w = Uniform live-load, lb. per sq.ft.;
W_b = Tofal weight of track and ballast or pavement on half-span, lb.;

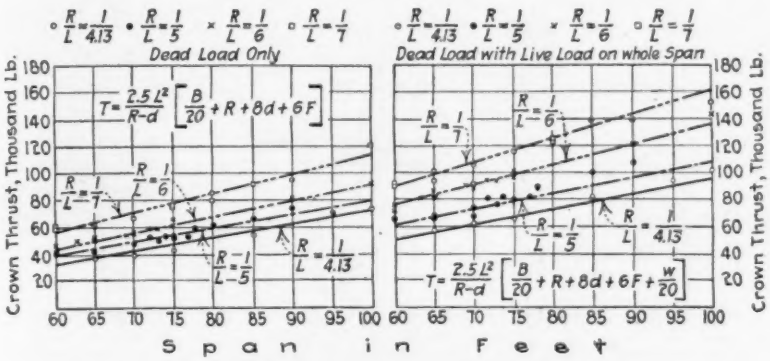


FIG. 2. CROWN THRUST BY FORMULA (LINES) AND IN ACTUAL ARCHES (POINTS)

W_f = Total weight of fill on the half-span, lb.;
W_c = Total weight of concrete on the half-span, lb.;
W_w = Total live-load on half-span, lb.;
T = Crown thrust for full load, lb.;
f_c = Maximum unit stress in arch ring, lb. per sq.in.

Consider a section of arch ring 1 ft. wide and assume the pressure line to pass through O at the crown and S at the springing, and take moments about S; then $T\,(R - d) = \Sigma M$, or $T = \dfrac{\Sigma M}{R - d}$, where ΣM is the sum of moments of the vertical loads or weights W_w, W_b, W_f and W_c about S. Now the values of W_w and W_b are respectively $\dfrac{wL}{2}$ and $\dfrac{BL}{2}$. The values of W_f and W_c

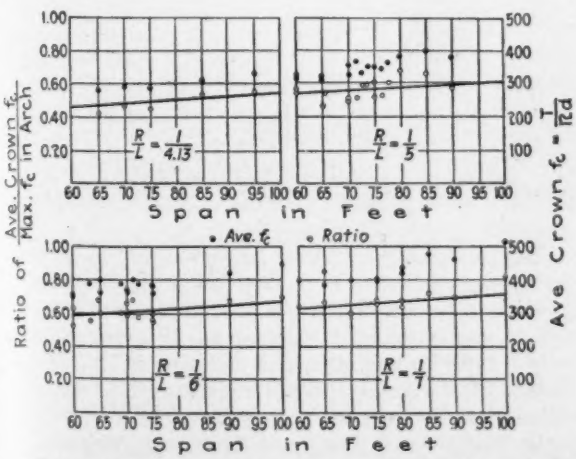


FIG. 3. RATIO OF MEAN CROWN STRESS TO MAXIMUM ARCH STRESS IN ACTUAL ARCHES

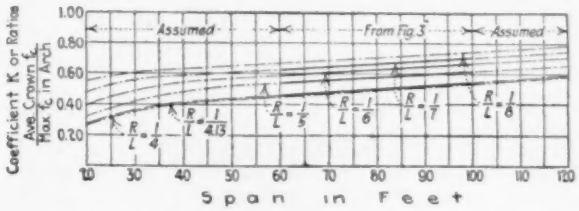


FIG. 4. DIAGRAM OF RECOMMENDED VALUES OF K FOR RAILWAY ARCHES

can be determined in terms of R, d, L and F by finding the areas of the fill and of the concrete, from the relation between the areas under the parabolas and certain rectangular areas, and then multiplying each area by its unit weight.

The area of the fill is equal to the difference between the area of the rectangle $BCGK$ and the area under the parabola BHK. The area of the concrete can be obtained by subtracting the area under the parabola AIM from the sum of the area of the rectangle $ABKM$ and the area under the parabola BHK. The area of the fill then is $(R - 2d + F)\,\dfrac{L}{2} - (R - 2d)\,\dfrac{L}{3} = \left(\dfrac{R}{6} - \dfrac{d}{3} + \dfrac{F}{2}\right) L$, and the weight of the fill is $W_f = \left(\dfrac{R}{6} - \dfrac{d}{3} + \dfrac{F}{2}\right) Lf$. The area of the concrete is $\dfrac{3dL}{2} + (R - 2d)\,\dfrac{L}{3} - \dfrac{RL}{3} = \frac{5}{6}dL$, and the weight of the concrete is $W_c = \frac{5}{6}dLc$.

From the same relations between the parabolas and rectangles the centers of gravities of the fill and concrete with respect to AC can be found by statical moments. The distance of the center of gravity of W_f from AC is

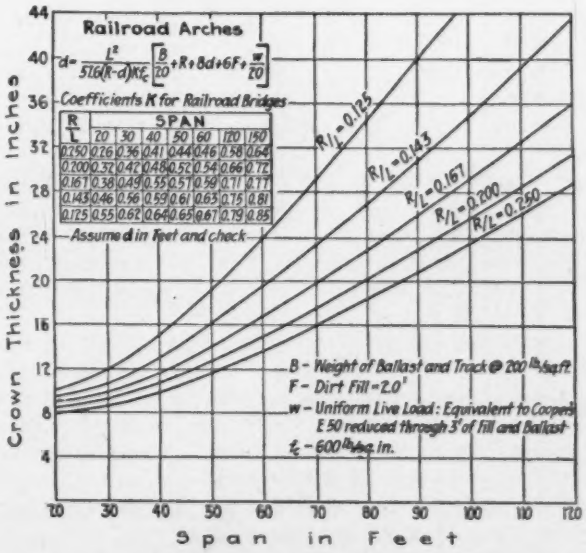

Railroad Arches
Coefficients K for Railroad Bridges
Assumed in Feet and check
B - Weight of Ballast and Track @ 200 lb/sq.ft.
F - Dirt Fill = 2.0'
w - Uniform Live Load: Equivalent to Cooper's E 50 reduced through 3' of fill and Ballast
fc - 600 lb/sq.in.
Crown Thickness in Inches
Span in Feet

FIG. 5. CROWN THICKNESSES AND STRESS COEFFICIENTS OF RAILWAY ARCHES

$$\frac{(R-2d+F)\frac{L^2}{8}-(R-2d)\frac{L^2 5}{48}}{\left(\frac{R}{6}-\frac{d}{3}+\frac{F}{2}\right)L}=\frac{\left(\frac{R}{48}-\frac{d}{24}+\frac{F}{8}\right)L}{\left(\frac{R}{6}-\frac{d}{3}+\frac{F}{2}\right)}$$

The distance of the center of gravity of W_c from AC is

$$\frac{\frac{3dL^2}{8}+(R-2d)\frac{L^2 5}{48}-\frac{RL^2 5}{48}}{\frac{5}{8}dL}=\frac{L}{5}$$

The centers of gravities of W_w and W_b are distant $\frac{L}{4}$ from AC.

With these values, ΣM in the expression $T=\frac{\Sigma M}{R-d}$ becomes

$$\frac{BL^2}{8}+\left(\frac{R}{48}-\frac{d}{24}+\frac{F}{8}\right)fL^2+\frac{dL^2 c}{6}+\frac{wL^2}{8}$$

and the expression for crown thrust finally is

$$T=\frac{L^2}{48(R-d)}[6B+(R-2d+6F)f+8dc+6w]$$

In general, the weight of fill is taken at 120 lb. per cu.ft., and of concrete at 150 lb. The value of the thrust T then becomes

$$T=\frac{2.5L^2}{(R-d)}\left[\frac{B}{20}+R+8d+6F+\frac{w}{20}\right]$$

That the assumptions made in deriving the expression for thrust are reasonable assumptions is shown in Fig. 2, by a comparison between the values of thrust according to the expression (shown by the curve) and actual values of thrust determined in a series of railroad arches analyzed according to the elastic theory (shown by the plotted points).

For the arches designed according to the assumed conditions of thickness and curvature, the relation between the maximum stress and average crown stress (determined by dividing the actual full-load crown thrust by the cross-section at the crown) is shown graphically in Fig. 3. The black dots in each case are the average crown stresses, or working crown stresses. The circles are the ratios of these average crown stresses to the maximum stresses anywhere in the arch. That some definite relation exists between these stresses is evident. The curves drawn through these circles may then be said to represent the variation of this ratio for the different ratios of rise to span.

The curves grouped in Fig. 4 provide a means for determining the value of the average crown stress f_cK, in terms of f_c, the assumed maximum stress in the arch. The heavier portions represent this relation as determined from the data and shown in Fig. 3. The lighter portions of the curves, from 100 to 120 ft. and from 60 to about 45, are obtained by producing these lines. A further extension of this relation to the very short spans would result in thicknesses less than would be desirable for practical reasons, and the variation is therefore arbitrarily changed to provide a general expression for all working conditions of railroad arches.

An expression for the thickness at the crown, in feet, may then be derived as follows:

$$d=\frac{thrust}{12\times 12\times average\ stress}$$

$$=\frac{T}{144f_cK}=\frac{2.5L^2}{144(R-d)f_cK}\left[\frac{B}{20}+R+8d+6F+\frac{w}{20}\right]$$

$$=\frac{L^2}{57.6(R-d)f_cK}\left[\frac{B}{20}+R+8d+6F+\frac{w}{20}\right]$$

In Fig. 5 is shown the application of the formula for crown thickness to a series of railroad arches for ratios of rise to span from ¼ to ⅛ and for conditions as noted

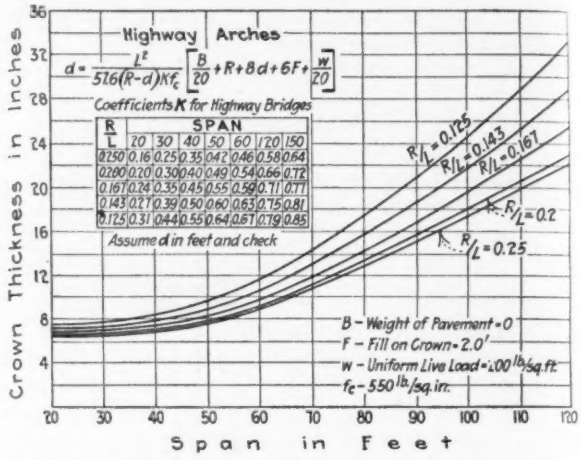

Highway Arches
$d=\frac{L^2}{516(R-d)Kf_c}\left[\frac{B}{20}+R+8d+6F+\frac{w}{20}\right]$
Coefficients K for Highway Bridges
R/L | SPAN 20 30 40 50 60 120 150
0.250 0.16 0.25 0.35 0.42 0.46 0.58 0.64
0.200 0.20 0.30 0.40 0.49 0.54 0.66 0.72
0.167 0.24 0.35 0.45 0.55 0.59 0.71 0.77
0.143 0.27 0.39 0.50 0.60 0.63 0.75 0.81
0.125 0.31 0.44 0.55 0.64 0.67 0.79 0.85
Assume d in feet and check
R/L = 0.125
R/L = 0.143
R/L = 0.167
R/L = 0.2
R/L = 0.25
B - Weight of Pavement = 0
F - Fill on Crown = 2.0'
w - Uniform Live Load = 100 lb./sq.ft.
f_c = 550 lb./sq.in.
Crown Thickness in Inches
Span in Feet

FIG. 6. CROWN THICKNESSES AND STRESS COEFFICIENTS OF HIGHWAY ARCHES

in the figure. For convenience the coefficients of stress for certain spans are arranged in tabular form. Intermediate values can be determined and the crown thickness for any other conditions easily obtained.

To adapt the formula for crown thickness to highway arches a slight change must be made in the value of coefficients for short spans, because of the light loads involved and because of a desire to obtain thicknesses which are practical. The coefficients for f_c for spans up to about 60 ft. therefore differ from the coefficients for the same spans for railroad arches. For spans over 60 ft. the coefficients are the same. A complete table of values for highway arches is given in Fig. 6, with an application to a series of highway arches for conditions shown in the figure.

To illustrate the application of the formula to a highway arch assume: Span, 90 ft.; rise, 12 ft.; weight of pavement, 100 lb. per sq.ft.; fill 1 ft.; live-load, 300 lb. per sq.ft. unit stress fc, 550 lb. per sq.in. Then k = 0.71. Assume depth at crown to be 23 in. = 1.92 ft. According to the expression,

$$Depth\ d=\frac{90\times 90}{57.6\times 10.08\times 0.71\times 550}[5+12.0+15.36+6+15]=1.91\ ft.=23\ in.$$

If the resulting thickness does not check the assumed thickness, another trial must be made.

In applying the formula for crown thickness it should be understood that f_c in the expression represents an approximate maximum value of stress which one may reasonably expect to obtain if the arch is designed according to the rules for curvature and thickness followed in the design of the arches here considered. The maximum value of stress for these arches varied from 575 lb. per sq.in. to 650 lb. per sq.in., with an average value of about 625 lb. per sq.in. A value of 550 lb. per sq.in. to 600 lb. per sq.in. for f_c is suggested for use in the formula.

Unlined Sewer Tunnel in Rock at Rochester

By Charles L. Raymond*

The Main and Front St. sewer tunnel in Rochester, N. Y., was driven at an average depth of 38 ft. below street level for a distance of 2,200 ft. along Main St. and 1,500 ft. along Front St.

Six working shafts, about 10x12 ft., were located in side streets varying from 20 to 40 ft. in distance from the main line of the tunnel. These shafts were connected by crosscuts to the main tunnel. Six angles were turned on the center line of the tunnel, and three different slopes were used in the grade.

For the engineering work a careful preliminary survey of all streets concerned was first made. Then a base or offset line 35 ft. south of the center line of Main St. and 19 ft. east of the center line of Front St. was located. Points were set on this base line on the center line of the shafts and crosscuts to the main tunnel. The shafts were then marked out and sunk to the required depth and from temporary points the crosscuts were driven over to the main line.

To carry the survey into the tunnel from the base line two 8-lb. plumb-bobs were hung in the shaft and carefully set in line on the surface. The transit was then taken down the shaft and into the crosscut. Here it was accurately set up in line with two plumblines and this line produced to permanent points set in the roof.

The survey points, in the tunnel roof, were spuds set in a wooden plug driven tight in a 2-in. hole drilled 4 in. deep. Both grade and line were carried on this spud. A theoretical roof grade was carried, and as each new point was set the contractor was given the distance the plug was above or below this grade; since the distance from the grade line to the bottom of the tunnel was constant it was a simple matter for the contractor to carry along the grade.

Some difficulty was experienced at first in keeping numbers on the plugs. Small tin tags were tried and then white paint, but the dust from the blasts and the corrosive effect of the tunnel water soon destroyed both. Then a heavy brass tag with the number stamped deeply into it was tried, and this proved satisfactory.

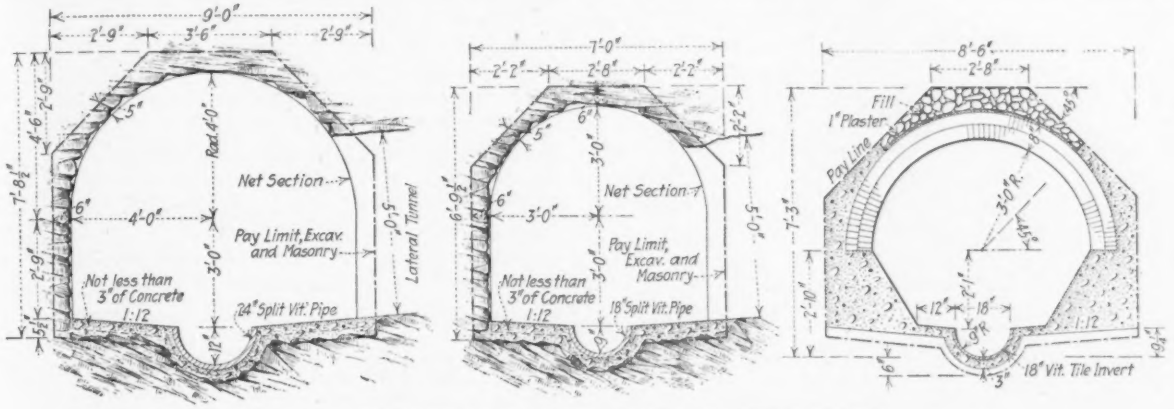


FIG. 1. SECTIONS OF MAIN AND FRONT ST. SEWER TUNNEL

FIG. 2. ROCHESTER SEWER TUNNEL DURING CONSTRUCTION, LOOKING WEST TOWARD ANGLE TO SHAFT 2

FIG. 3. LOOKING NORTH AT MAIN AND FRONT ST.

*Engineer in charge, Main and Front St. Sewer Tunnel, Rochester, N. Y.

All lines and grades carried into the tunnel were checked twice, and some three times.

The tunnel was driven both ways from all six shafts. The greatest error in alignment at the meeting of headings was ¼ in., and the error in grade was less than that. This being well within the allowable error the work was considered satisfactory.

The rock in the greater part of the tunnel was a good, hard limestone. The contractor had an electrically driven air compressor to furnish power for drilling. Pipe lines were laid along the curb of the street through which the tunnel was driven and carried into the tunnel through the working shafts. The Leyner type of drill was found to give the greatest satisfaction; "jackhamers" were used to a small extent in trimming the sides and bottom.

In drilling the heading from eight to ten holes, averaging 6 ft. in depth, were drilled, and these afterward were loaded with from four to six sticks of 60% dynamite. Holes were fired singly except the V-shaped cut in the center, which was fired first.

For the greater part of the work the ordinary fuse was used, it being cut to the required lengths for timing the shots. An average of 25 ft. a week was made in each heading, and at no time was traffic on the street delayed nor serious damage done to property.

There was very little over-breakage in the tunnel. Some occurred at Shaft No. 2 where the rock for about 250 ft. was quite soft, and also a little at Shaft No. 4 at the corner of Main and Front St. The rest of the tunnel had to be trimmed to meet the required dimensions.

Only a little trimming was necessary at Shaft No. 2. Here the roof of the tunnel was good rock, but the sides and bottom contained soft mud seams.

The excavation of rock has been completed, the last headings being holed through on March 3, 1916. The work now carried on is the laying of 18 and 24-in. vitrified tile invert and the placing of the concrete lining in the bottom, also the construction of brick manholes at all the shafts. These manholes will have larger cast-iron covers than ordinarily, for dumping snow.

These manholes are to be connected with all the existing sewers in Main and Front St. Concrete baffles are constructed in the manholes to prevent the sewage, in flowing down, from injuring the bottom.

⌘ ⌘

Cost of Factory Buildings of Timber and of Concrete

SYNOPSIS—The National Lumber Manufacturers Association has recently distributed as a bulletin a paper by F. E. Davidson, President of the Illinois Society of Architects, presenting a detailed estimate for the construction of two buildings on the same site—one of standard mill construction, the other of reinforced concrete. The paper shows that the cost of the latter building is about 30% more than that of the building of timber construction. The figures given by Mr. Davidson are of much interest; but since they are put forward by those interested in exploiting the use of wood in construction, it seemed desirable to present at the same time a statement of the case from the point of view of those who are well qualified to speak as to the merits of concrete for building construction. T. L. Condron, the designer of the concrete building referred to in Mr. Davidson's paper, has therefore prepared a statement which we print below, following Mr. Davidson's paper.

A Comparison of Concrete and Timber Factory Buildings

By F. E. Davidson*

I have as architect designed and superintended the construction of a great many manufacturing plants, both of fireproof construction and of the type known as standard mill. Based upon the actual construction costs, the increased cost of a strictly fireproof manufacturing building is from 25 to 35% greater than the actual first cost of an exactly similar standard mill building. In considering these costs I have included everything necessary absolutely to complete the building ready for the owner's occupancy; that is, the heating, lighting, elevators, power wiring, lighting fixtures, sprinkler equipment, window shades, window screens, etc., are all included in the cost of both types of construction.

*Architect and Engineer, Chicago, Ill.; President, Illinois Society of Architects.

It is of course manifest that the actual cost of many of the items of construction will be practically identical for either type of building. I have designed standard mill factory buildings in Chicago that have cost, complete, including all these various items, less than 7c. per cu.ft.; but I have never been able to secure the construction of a fireproof building, including all these items of cost, for less than 11c. per cu.ft., nor do I know of any accurate data showing any lower costs than those noted above.

DEPRECIATION CHARGES AND COST OF MAINTENANCE

When depreciation charges are considered, I have always recommended that a depreciation charge of 3½% per year be charged against a standard mill building and that a depreciation charge of 2½% per year be charged against an absolutely fireproof building.

As to maintenance charges: It is of course evident that as much outside painting, and in fact as much interior painting and calcimining, or whitewashing, will be required for a fireproof building as for a building of standard mill construction. Practically the only thing that wears and must be replaced in a mill building is the finished flooring, and I do not know of any accurate records of maintenance cost of a standard mill building which will average more than three-fourths of 1% per year. Yet, on the other hand, I do not know of any records of the maintenance charges on fireproof buildings which will average less than one-half of 1% per year.

INSURANCE RATING

As to insurance ratings: Every architect and engineer of course knows that a standard mill sprinklered building is given a much lower insurance rating than an absolutely fireproof building if unsprinklered. And even if the fireproof building is sprinklered, the difference in the actual ratings between a standard mill sprinklered factory and a standard fireproof sprinklered factory, with the ordinary occupancy, will only amount to about 10c. per $1,000 per year on the insurable value of the building.

ADVANTAGES OF STANDARD MILL CONSTRUCTION

Thus, considering first cost, depreciation, maintenance and insurance only, the argument is all in favor of the standard mill building. When I refer to a standard mill building I mean a building designed by a thoroughly competent industrial architect, who will inclose all stairs and elevators with brick masonry walls and have the openings to them protected with automatic firedoors, and will have all windows, with the possible exception of street frontages, of sheet metal and glazed with wired glass, with no unprotected vertical openings between any floors in the building. These are items that, more than any others, affect insurance ratings on any building.

There are in addition to the questions above noted some additional arguments in favor of the standard mill factory building. One of the principal ones that have recently been brought to the writer's attention is the fact that a skilled artisan will not work if compelled to stand on a concrete floor, if he can secure employment at equal wages in a standard mill building. Many owners of fireproof buildings have had trouble in keeping help, due to this reason only, and in some fireproof factories it has been necessary to cover the concrete floor with a layer of asphalt or to place cork carpet or wood thereon, in order to render the factory "habitable."

There is yet another argument in favor of the standard mill building, which in any large city should be given serious consideration, and that is the salvage value of the building itself. We all know that a standard mill building can be wrecked and, if the work is carefully done, all the structural material can be used in another structure, whereas in wrecking a concrete structure there is no salvage. In fact, I have been quoted by one of the largest contracting firms in Chicago a price for the wrecking of a monolithic concrete building which amounted to a trifle more than one-half the original cost of the building. Our American cities are growing so rapidly that it would indeed be a courageous investor, or at lease one blest with a prophetic vision, who could predict as to what particular use a certain piece of property would be best adapted for 25 or 30 years in the future.

A SPECIFIC CASE

The tables that follow represent the lowest proposals received by my office on a building to be erected for Walter E. Olson, of 42 South Laflin St., Chicago. Mr. Olson decided to build an addition to his present factory and at first purposed erecting the addition of reinforced concrete.

With this idea in view, plans and specifications were drawn for a concrete structure, the Condron Co., structural engineer, of Chicago, doing the engineering work. After these plans were made, the owner decided to find out what a standard mill building would cost him, with the result that another set of plans and specifications was prepared by Davidson & Weiss, architects, Chicago, and bids were taken on both sets of plans, which amounted to practically duplicating the contemplated concrete building in mill construction as nearly as it is possible to have uniformity of design between the two types.

The building is 55 ft. 1½ in. by 124 ft. 7½ in. in area, with six stories and basement, and contains 598,477 cu.ft. The story heights are in general 13 ft. 6 in. floor to floor. The typical bays of the building are 18 ft. by 17 ft. 10 in. The structure was designed for a live-load of 150 lb. per sq.ft. in accordance with the requirements of the Chicago building code, which limits the stresses in long-leaf Southern pine to 1,300 lb. per sq.in. in bending and 1,100 lb. per sq.in. in direct compression with the grain.

The floor girders are composed of two 10x18-in. timbers bolted together, and the floor joists or beams are 8x16 in., located 4 ft. 6 in. c. to c. The girders are carried on steel post caps of the writer's own design. The floor construction is a 3-in. tongued and grooved flooring finished with a ⅞-in. maple wearing surface.

This building is an addition to an existing factory, and it was necessary to use cantilever foundations for the entire structure. This, of course, is true for either design.

The timber specified in the contract was to be of the select structural grade as per the new grading rules of the Southern Pine Association. The cost to the contractor in this particular building at the site was as follows: Approximately $34.50 per M. for the 18-in. stock and $33 per M for the 16-in. stock.

ESTIMATE FOR STANDARD MILL BUILDING*

Excavation, concrete foundations and brick masonry complete	$15,884
Carpenter work complete	14,374
Miscellaneous concrete, including fireproof stairs	3,547
Structural steel and miscellaneous iron	7,150
Sheet metal	138
Cut stone	735
Roofing	145
Glazing	1,365
Steel sash	1,436
Firedoors	3,360
Fire escapes	1,479
Painting	720
Plumbing	615
Electric wiring	1,100
Plastering	90
Total	$52,138

* Exclusive of heating and sprinkler equipment.

On the basis of 598,477 cu.ft. the cost of the building complete, as shown from the total cost given above, exclusive of heating, sprinkler and architects' fees, is 8¾c. per cu.ft. In figuring the cubical contents the space from the bottom of the footings to top of parapet wall is included.

For the same building proposals were taken on the monolithic flat-slab concrete design, and the lowest proposals presented were as follows:

ESTIMATE FOR REINFORCED-CONCRETE BUILDING*

Foundations, reinforced concrete, masonry and carpenter work	$55,941
Sheet metal	138
Cut stone	735
Roofing	145
Glazing	1,470
Steel sash	1,936
Firedoors	3,360
Fire escapes	1,479
Painting	790
Plumbing	725
Electric wiring	1,250
Plastering	90
Total	$68,059

* Exclusive of heating and sprinkler equipment.

The cubic contents of the reinforced-concrete design on the same basis as cubic contents for the timber design equal 585,467 cu.ft. Thus the total estimated cost of building, exclusive of heating, sprinkler and architects' fees, would be 11½c. per cu.ft., and the difference in first cost of the two designs, exclusive of heating, sprinkler and architects' fees, is $15,921.

The concrete design was made as inexpensive as possible and provided for the use of high-carbon steel reinforcing material; and in other ways costs were cut as much as possible, in spite of which fact there was a difference of 2¾c. per cu.ft. in the cost of the two types of construction.

In this instance it will be noted that the cost of the concrete structure would have been about 30% more than the cost of the mill building, and this saving to the owner was effected in addition to the advantages he gained in the matter of quick erection, etc. The insurance rate is the same on the standard mill building as it would have been on the concrete structure.

The Case for Concrete

Sir: The bulletin issued by the National Lumber Manufacturers Association, prepared by F. E. Davidson, has just been brought to the writer's attention. As our company is referred to as having prepared the plans for the reinforced-concrete building used in this comparison, we believe you will be interested in some later data from us on this very interesting subject, which show that for a factory building for the Imperial Brass Manufacturing Co. the bids received for the concrete construction ran only 17% above the cost for the mill construction for the same building, or only 4% more than the cost of the mill construction of the building cited by Mr. Davidson.

In all these comparisons the cost of heating, sprinkler and elevator equipment and plumbing fixtures has been omitted; and it is obvious that if the cost of these equipments, which may be assumed at 28c. per sq.ft. of floor surface for either type of building, be added to the cost per square foot shown by the bids received, the percentage of additional cost of concrete over mill construction will be reduced. Making this addition in all cases, the reinforced-concrete construction of the Imperial Brass Co. building is shown to be 14% more than for standard mill construction for the same building, or only 3% more than the cost of the mill building cited by Mr. Davidson.

These comparisons are even more instructive when it is noted that the Imperial building was designed for heavier live-loads per square foot than the Olson building cited by Mr. Davidson; and that the bids for the Imperial building for standard mill construction totaled 11% less per square foot of floor than in the case of the Olson building, showing that our design for standard mill construction meets the requirement for an exceedingly economical structure. Our designs for the concrete construction of both the Imperial and the Olson buildings were identical, so far as type and unit stresses were concerned, and were for flat-slab, or girderless, floor construction. The accompanying table presents in ready form for comparison all the figures on which the foregoing comments are made.

It is perhaps well to state that in the case of the Olson building, made the basis of the comparison by Mr. Davidson, the Condron Co. was engaged as consulting engineer by Mr. Davidson, architect, to prepare the general design for the reinforced-concrete construction of the proposed Olson building, and Mr. Davidson prepared the architectural plans, including the general layout for both the concrete and the mill buildings.

The specifications and general instructions to bidders for both constructions were issued by Mr. Davidson. After the concrete plans were started and fairly well advanced, but before bids were called for, the architect informed us that the owner had decided to abandon the idea of a concrete build-

COMPARISON OF COSTS OF MILL AND CONCRETE CONSTRUCTION

Comparison of Bids for a Factory Building of Standard Mill and Reinforced Concrete	Olson Building		Imperial Building	
	Mill	Concrete	Mill	Concrete
Type of construction				
Total floor areas, sq.ft.	44,500	45,000	74,000	74,000
Masonry (brick, stone and concrete)	$20,256	$56,766	$31,097	$80,000
Ornamental and miscellaneous iron, etc.	11,989	4,839	13,250	
Carpentry	14,374	.....	23,500	.....
Steel sash, glazing, painting, roofing, etc.	3,804	4,479	6,069	6,883
Plumbing (drainage)	615	725	1,536	1,669
Wiring*	1,100	1,250	1,830	2,060
Totals of bids received	$52,138	$68,059	$77,282	$90,479
Per sq.ft. of floor areas	$1.17	$1.51	$1.04	$1.22
Relative costs per sq.ft.	100%	129%	89%	104%
Relative costs per sq.ft.	.....	.....	100%	117%
Adding 28c. per sq.ft. of floor area to cover cost of sprinkler, heating and elevator equipment and plumbing fixtures	$1.45	$1.79	$1.32	$1.50
Relative costs per sq.ft.	100%	124%	91%	103%
Relative costs per sq.ft.	.....	.....	100%	114%
Per cu.ft. of building	8¼c.	11¼c.	8c.	9¼c.
Relative costs per cu.ft.	100%	132%	94%	107%
Relative costs per cu.ft.	.....	.....	100%	114%
Adding 2c. per cu.ft. of building to cover cost of sprinkler, heating and elevator equipment and plumbing fixtures	10¼c.	13¼c.	10c.	11¼c.
Relative costs per cu.ft.	100%	126%	95%	106%
Relative costs per cu.ft.	.....	.....	100%	112½%

*Estimates of wiring only.

ing and to build of "mill construction." We suggested that since the concrete plans were so far advanced it would be interesting and perhaps instructive to get comparative bids.

This was done, and the results have proved interesting; but we think they are not especially instructive. The column spacing was modified in making up the mill design, changing the spans from 21 ft. 0 in. (concrete design) to 17 ft. 10 in. (mill design).

At the time the bids were called for on the Olson building, the plans for the Imperial Brass Co. building were underway in our office, and in this case the owner desired bids on both concrete and mill construction. The Imperial building was to be an addition to a standard mill building located in the same neighborhood as the Olson building. After the bids were received the owner decided to have the addition built of standard mill construction, notwithstanding the small difference in cost. Doubtless one of the factors in arriving at this decision was the greater ease in tearing down a mill building, provided it was found desirable to change the character of the improvement within a comparatively few years.

Attention has been called to the objection of skilled artisans to working upon concrete floors instead of wooden floors. As you doubtless know, many reinforced-concrete buildings are provided with maple wearing surfaces to obviate this very objection, but it is worth while to remember in this connection that the Ford Motor Co., with several hundred acres of floor space devoted to factory purposes and employing many thousands of skilled artisans, has adopted concrete construction for all its buildings and has used on all its later buildings cement finish floors rather than wooden wearing surfaces. This is also true of practically all the automobile factories throughout the United States, and there seems to be a growing preference for concrete floors over wooden floors, especially in machine shops. The trouble with working on concrete floors is largely imaginary in buildings that are heated. It is true that, in unheated buildings, stone and concrete, being better conductors than wood, are cold to the feet and therefore unpleasant to work upon.

The equipment of buildings with sprinklers seems to the writer to be quite as important in concrete construction as in mill construction, because the contents of buildings are usually not fireproof, even if the building construction itself is fire resisting. We have seen serious conflagrations spread throughout so-called fireproof buildings, fed only by the inflammable contents. In these cases the fires would have been almost insignificant had the buildings been equipped with sprinklers.

With reference to the "prophetic vision" necessary to know what kind of building will be required in any locality in the future there is no question that a first-class reinforced-concrete building six or eight stories high with wide column spacing will serve any and all purposes for which buildings may be required so long as men carry on pursuits in structures protecting them from the weather.

The Imperial Brass Co. building that has been referred to, which is now under construction, is 85x125 ft. in area, six stories and basement, and has 74,000 sq.ft. of floor surface. The cubical contents measured in the same manner as adopted for the Olson building are 972,000 cu.ft. for the standard mill construction and 993,400 cu.ft. for the reinforced-concrete construction, the difference in cubical contents being due to the greater depth of foundations for the concrete construction. The typical bays of the standard mill construction are 18 ft. by 16 ft. 6 in. and for the reinforced-concrete construction 18 ft. by 21 ft. 6 in. The structures were designed for a live-load of 175 lb. per sq.ft. in accordance with the requirements of the Chicago building code.

In the standard mill construction the floor girders are composed of two 8x18-in. timbers, bolted together, and the floor joists or beams are 8x16 in. spaced 4 ft. 6 in. c. to c. The girders are carried on steel post caps built up of plates and angles. The floor construction is 3 in. tongued and grooved yellow-pine flooring with ⅞-in. maple wearing surface. The timber specified was select structural-grade Southern pine, according to the specifications for dense Southern pine given in the Southern Pine Association Density Rule Book of Mar. 15, 1916. No. 1 Douglas fir was permitted as an alternate for the above yellow pine.

The reinforced-concrete design called for an 8-in. reinforced-concrete slab supported by flaring column heads, and reinforced-concrete round columns.

The foundations for the mill-construction building are the usual spread type, except that cantilever foundations are required on the side adjacent to the old building.

CONDRON CO.,
By T. L. CONDRON.

Monadnock Block, Chicago, Oct. 21, 1916.

Southern Rains and Floods of July, 1916, Exhibited

The destructive floods of July, 1916, in the East Gulf and the South Atlantic States are the subject of a valuable special study by Prof. Alfred J. Henry in the *Monthly Weather Review* for August, published Nov. 1. Through the courtesy of C. F. Marvin, chief of the Weather Bureau, we are able to present material from advance proofs of the study and also to announce that "a small number of separates will be available for distribution to engineers who may be particularly interested in the conditions that produced the flood."

The floods were caused by two tropical cyclones. The first of these occurred during the night of July 5-6, passing inland through Mississippi, and was accompanied and followed by almost continuous rain for five days, with extraordinary precipitation in some sections. The second cyclonic storm passed inland over South Carolina the morning of July 14. Before this storm died out, "unprecedented rains fell over the eastern slope of the Blue Ridge and also in the valley of the French Broad." The saturated condition of the soil contributed to a probable 80 to 90% runoff and unprecedented floods in both the Tennessee and Atlantic drainage areas.

The study by Professor Henry includes detailed rainfall figures for the two storms for many stations and also 20 maps and diagrams, mostly rainfall maps, but including a few hydrographs or river-elevation diagrams. Four rainfall maps of most general interest are reproduced here as Figs. 2 to 5, and some cumulative rainfall diagrams as Fig. 6.

Fig. 2 illustrates an "island" of heavy precipitation in southeastern Mississippi, the most notable of three such islands in the storm of July 5 to 8. Professor Henry states that there is no explanation of these islands.

Taking the whole area covered by the storm of July 5 to 8, 1916, the records show that during a single 24-hr. period more than 8 in. of rain fell on an area of 4,945 sq.mi. and more than 4 in. over 17,582 sq.mi. High records in three states for 24 hr. were: 12.7 in. at Alaga, Ala.; 10.8 at Eufala, Ala.; 9 at Ozark, Ala.; 9.9 at Blakely, Ga.; 9 at Fort Gaines, Ga.; and 10.8 in. at De Funiak Springs, Fla.

Taking up high records for the second of these cyclonic storms, farther east, from 8 to 10 in. of rain was recorded

July 14 at five North Carolina stations. The climax occurred later and is thus summarized by Professor Henry:

> Beginning during the afternoon of July 15 and continuing for 24 hr. the rains in the mountain districts of North Carolina were extraordinarily heavy. At a special orchard station maintained at Altapass in the southeastern corner of Mitchell County, N. C. (altitude 2,625 ft. above mean sea level), an actual measurement of 19.32 in. of rain in 24 hr. was recorded in an 8-in. standard rain gage. The measurement was made at 6 p.m., July 16, 1916. According to the observer, J. S. Bowen, between 2 p.m. Saturday and 2 p.m. Sunday, July 16, the rainfall amounted to 19.32 in. as measured on the 16th, and about 2.90 in. of the rainfall measured on the afternoon of the 15th fell after 2 p.m. of that day; hence the 24-hr. rainfall, 2 p.m. 15th until 2 p.m. the 16th, was about 22.22 in.

The highest hourly rate of precipitation mentioned by Professor Henry is 5.21 in. per hr. at Mobile, Ala., July 8, 1916. This continued for 25 min., "and appears to be the greatest fall for 25 min. in August" ever recorded on an automatic gage, though a rate of 6.36 in. continued for 20 min. was visually observed at Jupiter, Fla., Aug. 21, 1893. Details of hourly rates for July 5 to 8, 1916, are given in the report for Mobile and Montgomery, Ala., Meridian, Miss., Birmingham, Ala., Atlanta, Ga., Chattanooga, Tenn., and Asheville, N. C.

The cumulative precipitation for six stations, on as many rivers, is shown graphically by Fig. 6.

Flood conditions are reviewed by Professor Henry. As the storm progressed upstream, "we have the rather anomalous situation of a river being in flood in its lower reaches before the headquarters have reached the flood stage." Also, "the flood stage was naturally long-drawn-out by reason of the advance of the rains in a direction directly opposite to that of the course of streams." High-water marks were greatly exceeded at many places.

The exact loss of life as a result of the floods caused by the two storms is unknown, but is placed at 80 persons, most of whom were drowned in the streams of North Carolina. The estimated property loss totals $21,724,000, divided as follows: Buildings, roads, bridges, culverts, etc., $4,918,000; crops not gathered, $11,606,000; live-stock and movable farm property, $811,000; suspension of business, loss of wages, etc., $1,939,000; railroads, in roadbed, bridges, trestles, culverts, etc., $2,450,000.

As to the likelihood of similar storms in the future Professor Henry says:

> Unfortunately, our present knowledge of the underlying causes of cyclonic storms, their distribution in time and space, is so indefinite that any discussion thereon must be largely speculative. It may be said, however, that the floods in Mississippi, Alabama and Georgia were due almost wholly to a single long-continued rainstorm which was closely associated with the passage inland of a tropical cyclone. Likewise, the floods of the rivers in the coastal plain of South Carolina, especially the Great Pee Dee, were due to the movement inland of a single tropical cyclone. The floods in the streams originating in North Carolina were due to the torrential rains of the 15th and 16th coming at the close of a period of heavy rains that were associated with the first, or Alabama, storm. Tropical cyclones do not, as a rule, synchronize as did these two, and on that fact we would base our belief that a repetition of the storms of July, 1916, is not probable more than once in a century at least.

FIG. 1 - PATHS OF HURRICANES AND OVERFLOW AREAS ALONG SOUTHERN RIVERS

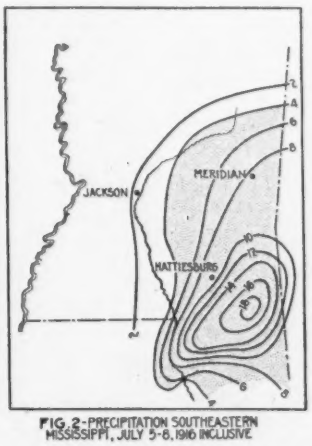


FIG. 2 - PRECIPITATION SOUTHEASTERN MISSISSIPPI, JULY 5-8, 1916 INCLUSIVE

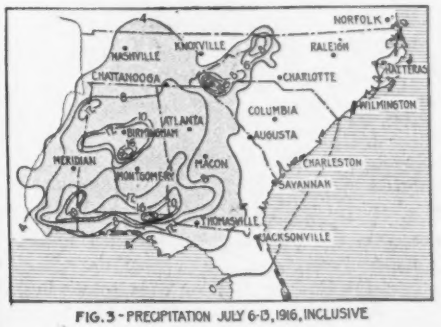


FIG. 3 - PRECIPITATION JULY 6-13, 1916, INCLUSIVE

FIG. 4 - PRECIPITATION JULY 15, 1916

FIG. 5 - PRECIPITATION JULY 14-18, 1916, INCLUSIVE

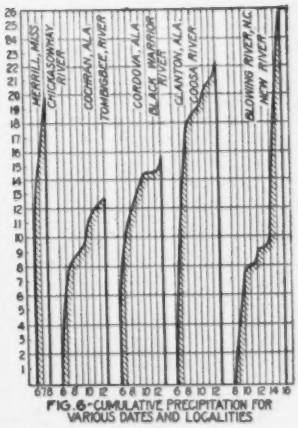


FIG. 6 - CUMULATIVE PRECIPITATION FOR VARIOUS DATES AND LOCALITIES

RAINFALL IN THE EAST GULF AND SOUTH ATLANTIC STATES DURING TWO CYCLONIC STORMS IN JULY, 1916

It seems hardly necessary to advise those desiring the full paper on these great storms to lose no time in applying to the Weather Bureau, Washington, D. C., for a copy.

Electric-Automobile Fire-Fighting Equipment has been generally adopted by the City of Paterson, N. J., replacing horse-drawn apparatus. After a study of motor apparatus in use elsewhere, it was decided that the readiness hill-climbing power, simplicity of operation and general reliability dictated use of electric traction in that city. It is reported that there will be required six tractor-drawn (two-wheel front drive) steam pumping engines, five combination chemical engines and hose wagons and one four-wheel-drive aërial truck. Battery-charging apparatus will be installed in each engine house.

Six-Post Water-Tower Tank

SYNOPSIS—The design and erection at Wheaton, Ill., of a steel water tower and tank having a total height of 173 ft. are described.

An elevated water tank of 200,000-gal. capacity, with a six-post steel tower, is being built for the municipal water-works at Wheaton, Ill., to replace an old 138,000-gal. standpipe. The tower is 138 ft. high from the concrete footing to the horizontal ring girder, and the height to the top of the tank is 172 ft. 6 in. The tank is 28 ft. in diameter and 35 ft. high in the cylindrical portion. It has a segmental bottom 14 ft. deep, connected to a 48-in. riser pipe, and is covered by a conical steel roof.

The tower has a batter of 1 in 12 and is divided into four stories or bays by three courses of horizontal struts. The columns are of box section, each composed of two 15-in. side channels and an inner 18-in. cover plate, with double lacing on the outer side. They are made in four lengths of about 38 ft. The horizontal struts are pairs of channels with top and bottom lacing, except that at the ends the channels are brought together and riveted to connection plates on the columns. Each panel has two adjustable diagonal tension rods, set up by turnbuckles and having welded loop ends fitted to 2-in. pins in the connection plates. The loops are double, straddling the plate and engaging both ends of the pins. The connection plates at the bottom are on the outside of the columns; the others are riveted to the sides—that is, to the webs of the channels.

Near the tops of the columns are outside brackets to carry the ring girder, above which the column is extended about 8 ft. by four 6x4-in. angles, which are riveted to the bottom course of the side of the tank. Large reinforcing plates are placed between the backs of the columns and the tank plates.

The tank has five courses, 7 ft. high, with each course outside of the next course below. The plates are 87 in. high and 15 ft. long, with double riveting in the vertical seams and single riveting in the horizontal seams. All seams are made with lap joints. The thickness is $\frac{3}{8}$ in. for the segmental bottom and the lowest course of plates, $\frac{5}{16}$ in. for the second course and $\frac{1}{4}$ in. for the other courses. At the top are brackets carrying a circular channel supporting the 5-in. channels forming the rafters of the conical roof, which has $\frac{1}{8}$-in. plates. The roof projects about 18 in. beyond the tank and fits close against its upper edge, so that there are no openings by which birds can enter.

At the base of the cylindrical portion of the tank is a ring girder with a $\frac{1}{4}$-in. horizontal web plate 41 in. wide, having radial stiffeners on the under side. The inner flange is a single angle riveted to the tank shell; the outer flange is a 6-in. channel having its lower flange riveted on top of the web plate. The hand railing around this girder consists of steel angle stanchions and top rail, with two diagonal bars in each panel.

About 18 in. below the top edge of the tank is an inside Z-bar ring that serves as a stiffener. On the cover is a swivel trolley for a painters' platform. The top of it is attached to the apex of the roof in such a manner as to allow it to revolve, and its lower end supported by means of a pair of $3\frac{1}{2}$-in. flat-tired wheels between a pair of angles. This frame carries an outside bent plate having a hole for a clevis or stirrup to which the tackle of the painters' platform is hooked. The overflow is a 3-in. pipe leading out through the top course of the tank and carried down one of the columns.

The 48-in. riser is of $\frac{1}{4}$-in. plates in $5\frac{1}{2}$-ft. courses, with single-riveted circular and vertical seams. At the top is a domed cover with a telescopic expansion-joint connection to the 12-in. pipe entering the tank. The bottom is flat, with a 12-in. opening for the pipe connection to the mains and a 6-in. opening for the pipe

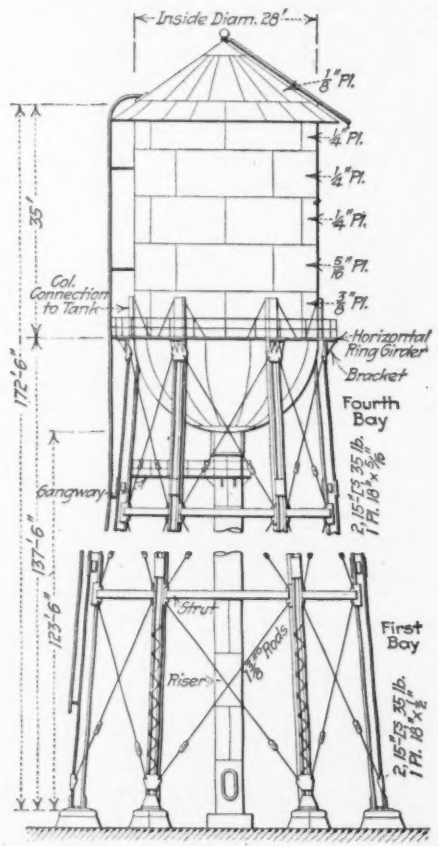


FIG. 1. DESIGN OF SIX-POST WATER TOWER WITH 200,000-GAL. TANK

from the service pumps. A 10x16-in. manhole is provided in the lowest course of plate. With the large body of water in a riser of this size no frost-protection covering is needed.

To give access to the expansion joint, there is a gangway from the column carrying the ladder to the riser pipe. Around the riser is a horizontal ring girder 4 ft. wide, supported by brackets on the riser and forming a platform. The gangway has a floor plate $16\frac{3}{4}$ in. wide, resting on the bottom flanges of a pair of channels and stiffened by transverse angles across the bottom of the plate and channels. Hand-rail stanchions are riveted to the backs of the channels. In this gangway, and in the ring girders and the Z-bar stiffener, holes are made in the horizontal web plates to provide drainage.

The foundations consist of six heavy concrete footings or piers for the columns, and a central pier for the 4-ft. riser. The site is on a good gravel formation, and the pressure of the base of the piers upon the soil averages 2,770 lb. per sq.ft. The column piers are about 8½ ft. high, 5x5 ft. at the top, which is inclined so that the columns will be perpendicular to it. In each pier there are two 1⅞-in. anchor rods, with 2¾-in. upset ends. The anchor rods are inclined 1 in 12 and have the upper ends incased in 3-in. pipe sleeves 24 in. long, into which the rods are grouted after final adjustment. The size of the piers was not governed by the bearing, but was made to comply with a specified factor of safety of 1½ on uplift.

The central pier is 8 ft. square and 9 ft. deep. It has a chamber for the pipe connections and a valve on a 6-in. blowout pipe. The top of the pier, over the chamber, is reinforced with steel rods. There are 221½ cu.yd. of concrete in each column pier and 18¾ yd. in the central pier. The concrete is a 1:3:5 mix (made with stone), except that for the caps of the column piers it is 1:2:3. It was made in a mixer of the continuous type, and the concrete for each pier was poured in a continuous run.

The tower and tank are of openhearth steel conforming to the specifications of the American Railway Engineering Association. The live-load on the roof is taken

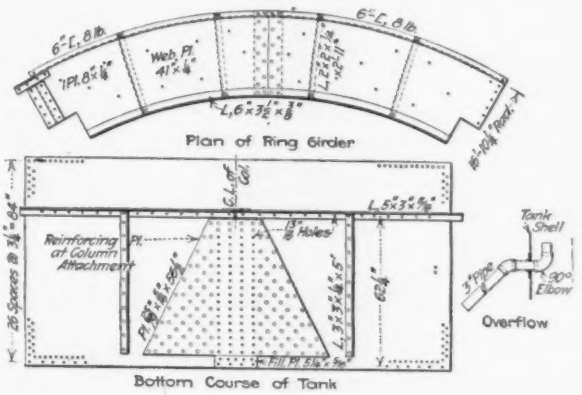


FIG. 3. RING GIRDER AND BOTTOM COURSE OF TANK SHELL
Showing reinforcing plate at column connection

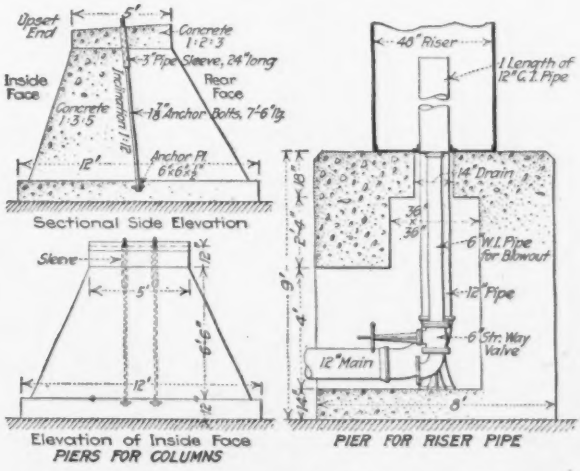


FIG. 4. FOUNDATION PIERS FOR WATER TOWER

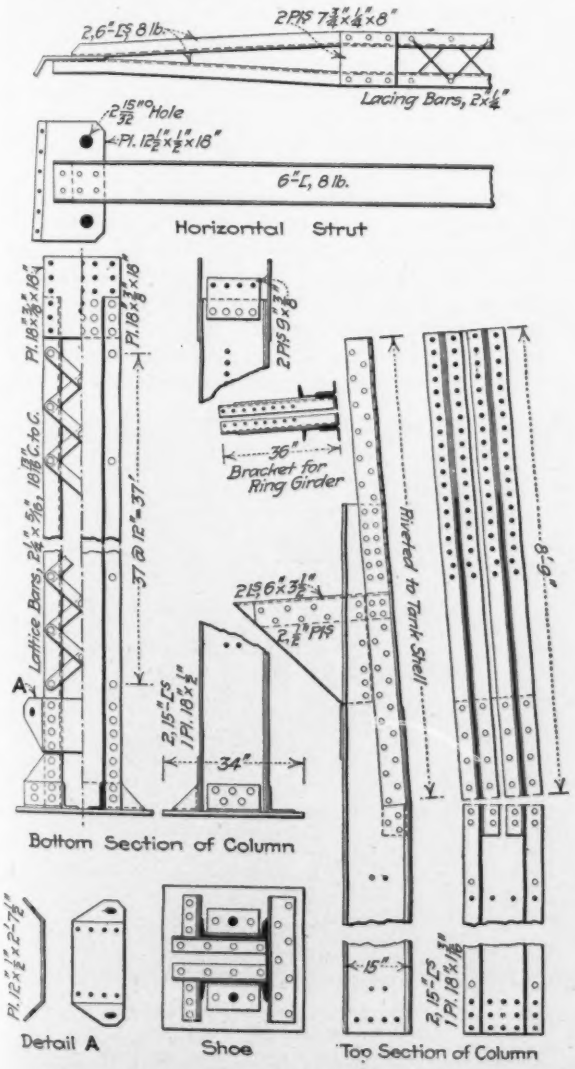


FIG. 2. DETAILS OF COLUMNS AND STRUTS OF WATER TOWER

at 30 lb. per sq.ft., or 200 lb. concentrated at any point. The wind pressure is taken as 30 lb. per sq.ft., acting on 66% of the diameter of the tank for its full height. The steel was given one coat of paint at the shop and one field coat after erection.

Erection of Water Tower

The erection outfit was very simple. A light timber A-frame sufficed to raise the tower. It was just high enough to enable the tackle to be hitched above the middle of a column length, the column thus swinging free and being guided into position by guy lines. For the upper bays the A-frame was placed upon each line of struts in turn. With the tower completed, including the ring girder and the bottom course of the tank (to which the heads of the columns are attached), the next step was to place the segmental bottom. Two or three rope tackles were attached to the top of the tower, operated by a team or a hand winch for heavy hoisting, and by hand for adjustment of the parts.

Two of the bottom plates were hoisted into place and bolted to the tank shell, and then the bottom disk or circle plate (with pipe connection) was hoisted. With the aid of the tackles hitched to the circle plate and the ends of the two bottom plates the parts were brought together so that erection bolts could be fitted. The other bottom plates were then placed without difficulty. A platform composed of a few loose planks was used in this part of the work.

The tower and tank bottom were erected in this way without any riveting, all parts being connected by service bolts. The riveting was started on the column splices, working upward, and was completed before proceeding to finish the tank. All riveting was done by hand. The A-frame mentioned above was utilized to support the riveters' platform. It was placed horizontally with the wider end lashed to one of the struts and the narrow end attached to one of the ropes from the top of the tower. For riveting the tank seams a steel cage was used carried by long bars which were hooked over the top of the shell plates.

This structure was designed by L. J. Ruddock, City Engineer of Wheaton, Ill. It was built and erected by the Memphis Steel Construction Co., of Greensburg, Penn. The material was inspected at the shops by Robert W. Hunt & Co., of Chicago. The cost was $8,843 for the steel tank and tower and $1,071 for the concrete foundations.

Activated-Sludge Novelties at Hermosa Beach, Calif.

By A. M. Brosius*

SYNOPSIS—A diffuser wheel in the bottom of each tank draws air and sewage down through a central foam chamber to the bottom of the tank, from which both pass upward. Air under 1 lb. pressure will also be admitted to each central pipe.

Among the assets of the cities and towns on the southern California coast are the fine bathing beaches. It is therefore unadvisable, for æsthetic reasons, to discharge sewage into the ocean adjacent to these beaches without its having been treated to a high degree.

On account of the most available site for the Hermosa Beach sewage works being located within the city limits, it was necessary to select a method of treatment that would be unobjectionable to the near-by property owners, and the activated-sludge process was recommended.

In view of the fact that the porous plates used as air diffusers in some of the activated-sludge plants have caused more or less trouble by clogging with oil and dust, an effort was made to develop a method of air diffusion that would obviate this difficulty, and a series of experiments covering a period of several months were made for this purpose.

The experimental plant (Fig. 1) includes a circular wooden tank, 5 ft. in diameter, with an available water depth of 9½ ft. This tank has glass portholes at 1, 2, 4 and 5 ft. above its bottom. Diffusers of various designs were tried out, both with and without compressed air being supplied to them.

The experiments resulted in the adoption of a diffuser wheel, similar to an impeller of a centrifugal pump, revolving on a vertical shaft. This wheel is in the bottom of the tank, 4 ft. from one side, and at the lower end of a vertical galvanized sheet-iron pipe, 7½ in. in diameter, which extends about 3 in. above the surface of the water when the depth of the water is 9½ ft. In the side of the pipe is an opening 6 in. square, the center of which is 1 ft. under water. This opening is provided with a gate so that the amount of water entering the pipe can be controlled. It was found that the water as it enters the pipe forms a foam with the air in the top of the pipe, which may be designated as the foam chamber. The action of the impeller draws this foam to the bottom of the pipe and throws it out approximately horizontally, thereby diffusing the air uniformly throughout the contents of the tank, the diameter of the bubbles ranging from ⅛ to $^1/_{100}$ in. or smaller.

It is believed that under ordinary conditions of sewage flow sufficient air will be drawn down the vertical pipe without supplying air under pressure. It was considered advisable, however, to provide a blower for the Hermosa Beach plant (Fig. 2) for furnishing air to the foam chamber under a pressure of about 1 lb. This was done in order that the air may be more readily measured. The air pressure is maintained practically uniform by a trap

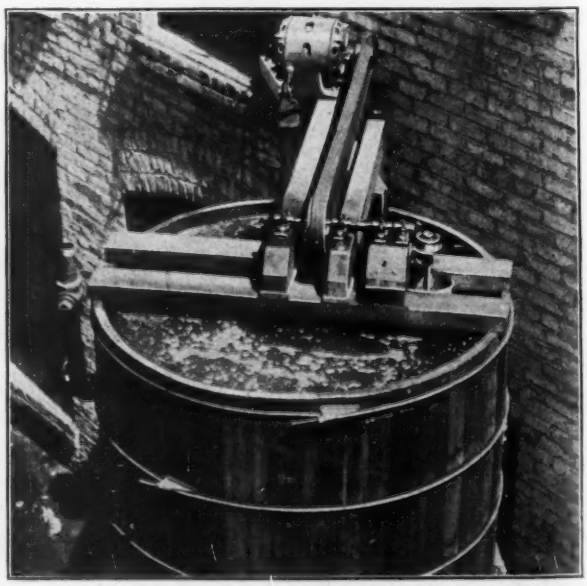


FIG. 1. EXPERIMENTAL AIR-DIFFUSER TANK AT HERMOSA BEACH, CALIF.

The tank contains clear water. The only air diffused through it is being drawn down the vertical pipe and thrown out by the centrifugal action of the diffuser wheel in the bottom of the tank

*Consulting Engineer, Hollingsworth Building, Los Angeles, Calif.; formerly Principal Assistant Engineer, Baltimore Sewerage Commission.

in the sewage inlet channel, which enters one side of the foam chamber, and by an elbow pointing downward, which enters the foam chamber on the opposite side, the head on the lower end of the elbow being slightly less than the head in the trap.

The operation of the Hermosa Beach plant may be briefly described as follows: After the sewage passes one of the first pair of tanks adjacent to the machinery room. There are four pairs of tanks 12½ ft. in diameter, with a maximum water depth of 12 ft. The tanks adjacent to the machinery room may be used for activating the sludge, or as aërating tanks.

On the assumption that the sludge has been properly activated in one tank, the sewage, mixed with air, is discharged into it at the bottom of this tank by the diffuser wheel. The revolution of the wheel gives the mixed sewage and air a high velocity as it leaves the wheel. The diffuser wheel also gives a rotary motion to the entire contents of the tank, which causes the air bubbles to rise in a spiral path having an angle of about 45° with the vertical. The ratio between the length of the spiral path of the bubbles and a vertical path is approximately 14 to 10. This means that the spiral path of the bubbles in a tank 10 ft. deep would be about 14 ft. long, thereby giving a proportionately longer period of contact between the bubbles and the sludge.

The sewage leaves the tank by a circular weir-like overflow basin in the center of the tank, at the top, and runs through an inclined pipe leading to a double Y-trough between the second pair of tanks, where it may be diverted either in whole or in part to either one, or both, of these tanks. This diversion is accomplished by means of two swinging tongue-like gates in the double Y-trough.

The sewage, upon entering the second pair of tanks, passes into the foam chambers, mixes with air and is

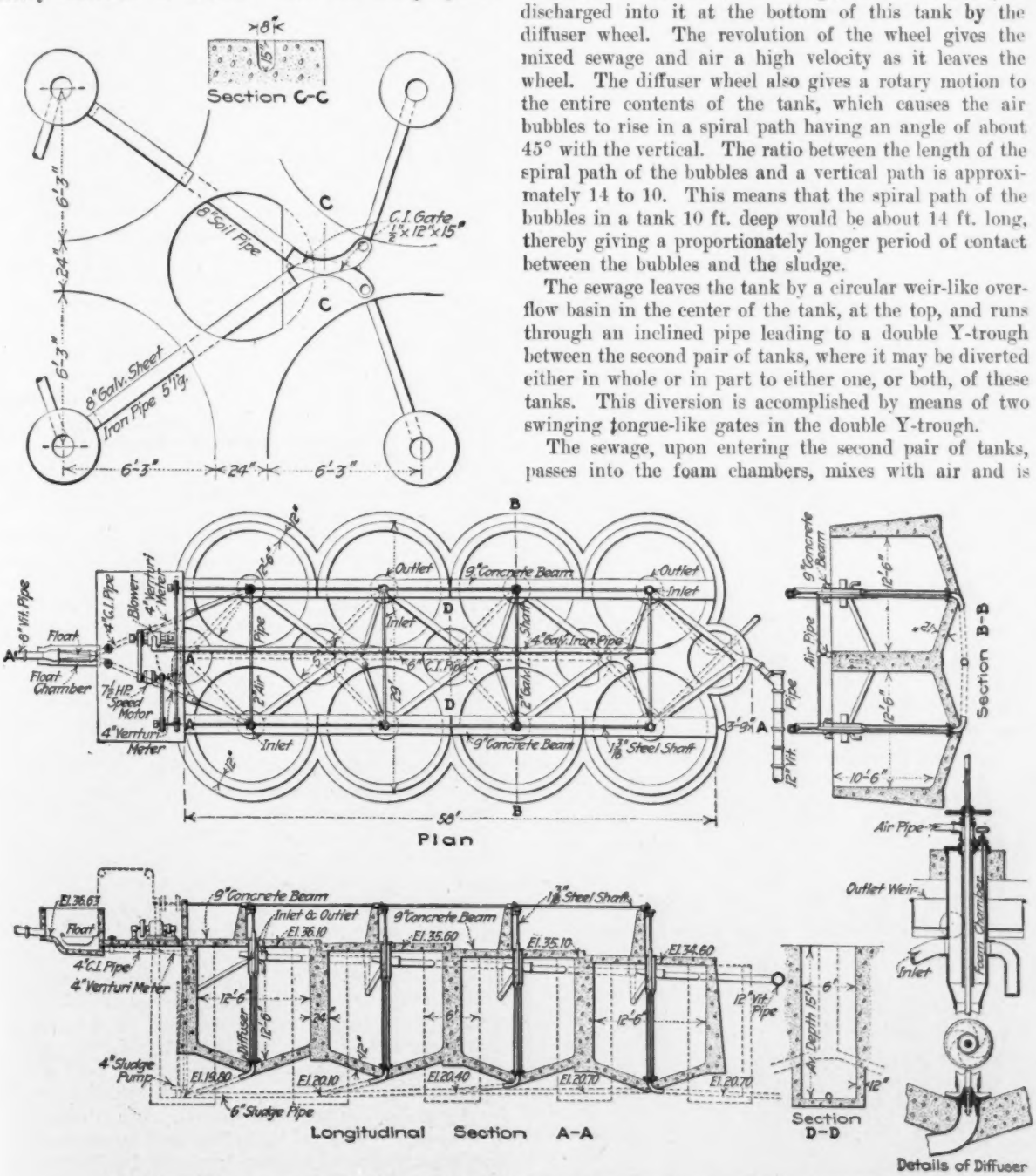


FIG. 2. DETAILS OF ACTIVATED-SLUDGE PLANT PROPOSED FOR HERMOSA BEACH, CALIF.

through grit and screen chambers, it enters a float chamber where the movement of a float is communicated through a cable to the brush-shifting device attached to a variable-speed motor. In this way the speed of the motor is controlled by the volume of sewage flow. The sewage on leaving the float chamber enters the foam chamber in thrown out by the diffuser wheel in the same manner as described in connection with the first tank. If the volume of sewage flow is large, it may be passed similarly through the third pair of tanks. If, however, the flow is small, it would be necessary to pass the sewage through only one or two tanks that are being supplied with air, the other

tanks acting as sedimentation tanks. When, however, it is necessary to use four tanks in addition to the first tank, the effluent from the last tanks would be discharged into one of the fourth pair of tanks, which would then operate as the sedimentation tank; and the clarified effluent from this tank would be discharged into the outfall sewer.

It may be desirable to oxidize further the sludge that has accumulated in one of the last pair of tanks. The other one of these tanks is then utilized as the sedimentation tank. The method of sludge aëration is accomplished in the sedimentation tanks by mechanism similar to that in the other tanks.

In the process just described most of the sludge in the first tank will be carried with the sewage through the several tanks and deposited in the final sedimentation tanks. From these tanks it may be discharged onto the sludge beds or returned to the sludge pump in a manhole between the first pair of tanks. The returned sludge is then lifted by the sludge pump and is discharged into one or the other of the tanks of the first pair, preparatory to repeating the cycle of operation.

While one of the first pair of tanks is being used for the purpose of sludge activation and when the sludge in the other tank is in proper condition, this last-mentioned tank would be used as the first aërating tank.

The bottom bearing for the shaft of the diffuser wheel is so designed that it serves as a valve on the upper end of the sludge pipe in each tank. This valve, together with the wheel and vertical pipe leading downward from the foam chamber, may be raised by means of a handwheel operating on the threaded upper end of the diffuser-wheel shaft. In this way the contents of any of the tanks may be drained out through the sludge pipes, whenever necessary, either for cleaning or for repairs.

The Hermosa Beach plant is designed for a maximum daily sewage flow of 400,000 gal., and an aëration period of 3 hr. It is estimated that the power required for a plant of this capacity will amount to about 60 kw.-hr. per 100,000 gal. of sewage treated. The writer's designs for this plant were approved on July 6, 1916, by C. R. Sumner, city engineer, and by the Board of Trustees of Hermosa Beach. It is expected that bids for construction will be received in October and the plant be in operation early in 1917.

The First Complete Railway-Valuation Reports Filed

Great interest attaches to the first complete reports of the valuation of railway companies' property by the Valuation Division of the Interstate Commerce Commission, which were filed at Washington on Oct. 9. The two roads of which the valuation has been completed are the Texas Midland R.R., a line 111 mi. long in Texas, and the Atlanta, Birmingham & Atlantic R.R. in Georgia and Alabama. The Federal Valuation Act required the Interstate Commerce Commission to ascertain for each railway the cost of reproduction new, the cost of reproduction less depreciation, and the original cost to date.

The principal figures for the Texas Midland R.R. under these three heads are as follows:

Road (wholly owned):	
Cost of reproduction new	$2,601,289
Cost of reproduction less depreciation	2,007,708
Equipment:	
Cost of reproduction new	582,071
Cost of reproduction less depreciation	362,940
General Expenditures:	
Cost of reproduction new	176,693
Cost of reproduction less depreciation	138,561
Total (including proportion of line owned jointly):	
Cost of reproduction new	3,382,004
Cost of reproduction less depreciation	2,527,417
Land:	
1,863 acres owned and used for transportation purposes	236,689
76 acres not held for transportation purposes and structures thereon	8,619
Original Cost:	
Original cost of road could not be found.	
Original cost of equipment now in existence	528,874
Original cost of 1,370 acres purchased	68,232
Original cost of 493 acres donated (cost to donors)	43,103

No other values or elements of value were found to exist.

The Texas Midland received aids and donations, including gifts of right-of-way, amounting to $69,885, not including contributions of Hetty R. Green.

The capitalization of the company since 1908 has consisted of $2,000,000 of 4% mortgage bonds and $112,000 of capital stock. The present capitalization of the road, therefore, is materially less than it would cost to reproduce the road in its present condition.

The railway was chiefly owned by the late Hetty R. Green, of New York City. Although commonly reputed to be the wealthiest woman in the United States, it is evident that it was not through such investments as that in the Texas Midland R.R. that she acquired her wealth.

The accounting division of the valuation force gives an extended history of the Texas Midland R.R.; and while it has not been able to ascertain from existing records all the data concerning the financial history, the principal facts may be briefly summarized as follows: In the early '80's the Texas Central R.R. built a line of railroad about 52 mi. long in central Texas. This, with other railway lines owned by the company, was sold under foreclosure proceedings in 1891. Mrs. Green owned 750 of the $1,000 bonds that had been issued by the Texas Central, and she took this 52-mi. line of road in exchange for her bonds. This line was incorporated as the Texas Midland R.R., and extensions were built with money furnished by Mrs. Green. Up to August, 1908, she had advanced to the company about $1,750,000. In addition, she had made other cash advances for freight cars and other requirements, making the total cash advanced to the company by Mrs. Green, according to the report of the accountants, $2,099,140. As far as the books disclose, Mrs. Green had up to June 30, 1914, never received directly or indirectly anything from the Texas Midland R.R., except a payment of $115,650 by the construction company that built the road, and interest on the bonds she held amounting to $334,596.

During the entire time from Feb. 26, 1893, when the railway company began business, until June 30, 1914, the entire net earnings of the company, above operating expenses and taxes, amounted to $1,127,000. From this amount, $354,000 was paid out in rents for land, buildings and rolling stock, leaving $773,000 balance available for payment of interest and dividends. The total amount of interest accruing on bonds during this period, however, amounted to $1,152,000, leaving a deficit of nearly $380,000, as the total operating results for that time.

Among other interesting facts found by the accountants in delving into the past history of the company is that when the Texas Central R.R. was sold at foreclosure proceedings in 1891 there were outstanding upon it mortgage bonds amounting to $3,399,000, of which Mrs. Green owned $750,000. The result of the foreclosure sale was that when the assets were distributed among the bond-

holders of the company the holder of each $1,000 bond received the sum of $151.37.

Atlanta, Birmingham & Atlantic R.R.

The Atlanta, Birmingham & Atlantic R.R. is a single-track narrow-gage railway about 634 mi. in extent, lying in Georgia and Alabama. It has been in the hands of a receiver since Jan. 1, 1909. The valuation results are summarized as follows:

Common-Carrier Property:

	Cost of Reproduction of Road and Equipment, Excluding Lands		Present Value of Lands
	New	Less Depreciation	
Owned by A., B. & A. R.R.	$22,716,886	$18,071,950	$1,091,886
Leased to others	170,754	111,366	13,668
Used by A., B. & A. R.R., owned	$22,546,132	$17,960,584	$1,078,218
Leased from others	1,608,866	1,448,226	1,213,195
Total used by A., B. & A. R.R.	$24,154,998	$19,408,810	$2,291,413
Noncarrier lands, present value			$145,203
Other Noncarrier Property:			
Stock in compress and terminal companies (par value)			4,505,000
Rails and track material			36,058

Aids, Gifts, Grants of Right-of-Way and Donations:

	Present Value
Carrier lands	$226,080
Noncarrier lands	21,795
Total	$247,875

In addition to the above, certain counties and municipalities donated sums of money amounting to $85,060 for the purchase of right-of-way. No other values or elements of value were found to exist, and the original cost of the property could not be found.

The various items entering into the physical-property valuation of the system are given as follows for that part of the system owned and used by the company:

	Cost of Reproduction of Road and Equipment	
1. Road:	New	Less Depreciation
Engineering	$589,503	$589,503
Grading	5,263,885	5,254,891
Tunnels and subways	46,856	43,575
Bridges, trestles and culverts	2,370,728	1,980,705
Ties	999,281	508,347
Rails	2,717,804	2,400,600
Other track material	539,920	372,990
Ballast	381,770	278,343
Track laying and surfacing	716,765	549,166
Right-of-way fences	11,254	5,929
Crossings and signs	173,975	125,529
Station and office buildings	635,614	470,155
Roadway buildings	338,227	239,282
Water stations	134,139	87,949
Fuel stations	58,413	28,497
Shops and engine houses	244,343	169,248
Wharves and docks	678,378	461,616
Telegraph and telephone lines	286	200
Signals and interlockers	7,050	4,825
Power-plant buildings	17,245	13,795
Power-transmission systems	2,715	2,243
Power-distributing systems	8,460	5,715
Power-line poles and fixtures	410	260
Miscellaneous structures	7,145	4,595
Roadway machines	36,751	22,504
Roadway small tools	9,276	4,636
Shop machinery	204,535	143,641
Power-plant machinery	27,001	21,531
Total	$16,221,729	$13,791,270
2. Equipment:		
Steam locomotives	$1,263,647	$942,795
Freight-train cars	2,895,376	1,570,641
Passenger-train cars	399,780	264,357
Work equipment	131,621	84,339
Total	$4,690,424	$2,862,130
3. General Expenditures:		
Organization expenses	$20,912	$16,729
General officers and clerks	125,473	100,378
Law	62,736	50,190
Stationery and printing	20,912	16,730
Taxes	104,561	83,649
Interest during construction	1,215,737	972,590
Other expenditures—general	83,648	66,918
Total	$1,633,979	$1,307,184
Grand total	$22,546,132	$17,960,584

The endeavor of the commission's accountants to untangle the past financial and corporate history of the Atlanta, Birmingham & Atlantic R.R. has not been very fruitful. For five of the companies whose lines make up the present system no records whatever could be found, and the records of the other three companies were incomplete. The accounts of the Atlanta, Birmingham & Atlantic R.R. itself were not so kept, moreover, as to make accurate statements possible.

The present system is the outcome of the schemes of certain Boston and Atlanta capitalists to build railway lines from Brunswick, Ga., to Atlanta and Birmingham, connecting with a steamer line from Brunswick to New York, to Boston. The railway was built, but the steamship line was never operated. The total outstanding capital liabilities of this company amount to $54,571,000, besides which $4,994,000 of receiver's certificates have been issued. Included in the capitalization is $25,000,000 of common stock and $10,000,000 of preferred stock.

As to the results of operations of the company, the accountants find that the total net earnings above operating expenses from the time the present company began operations in 1909 to June 30, 1914, were $4,190,000. During that time, however, rentals and interest on bonded debts have accrued amounting to $8,630,000, so that the company has not only paid no dividends on its stock, but since it has been operated has come short nearly $4,500,000 of earning its fixed charges.

⁂

Engineers as County Managers are urged by correspondents from California in letters published in the journal issued by the American Association of Engineers. One of these correspondents says: "Every county transacts its business through a board of supervisors or county commissioners, and 90% of the members of these boards are farmers who are unfamiliar with the principles of construction and management. The resulting waste is appalling. Members of the board are each engaged in his own private business and give attention to county affairs only at meetings occurring perhaps once a month. A good many thousand jobs will some day be given to county managers, and it behooves engineers and their friends to get busy."

⁂

The Pennsylvania Water-Supply Commission in its annual report states that the incorporation of companies to furnish domestic water-supply has almost ceased in Pennsylvania. Only nine such companies filed applications with the commission during the year, whereas during the five years prior to the creation of the State Water-Supply Commission from 100 to 150 such companies were incorporated annually, including companies for the development of water power. The commission received during the year 51 applications for the construction of dams and issued 43 permits. It received 29 applications for permits for alterations and repairs on existing dams. The commission's engineers conducted 626 field examinations on 348 existing dams and ordered repairs or alterations on 121 of these. The commission approved 485 applications for the erection of structures in stream channels, of which 402 were for bridges, 21 for fills and 62 for walls, piers, ice breakers, etc. The commission comments on the disastrous flood at Erie, Penn., on Aug. 3, 1915, in which 34 people were killed and $2,000,000 worth of damage to property was done, and says:

It is believed that this flood would have passed off without unusual damage had it not been for the restricted condition of the channel through the city. The commission had previously called the attention of the city officials to the obstructed condition of the stream and recommended that a comprehensive plan for its improvement be prepared and submitted to the commission for approval, but nothing was done until after the catastrophe had occurred.

The commission urges legislation empowering the commission to establish encroachment or shore lines along streams in densely populated districts to serve as permanent boundary marks within which no encroachments may be placed and requiring where possible the removal of existing encroachments. The commission is ready to proceed with the construction of the Pymatuning reservoir to regulate the flow of the Shenango and Beaver Rivers as soon as the necessary funds are provided. An appropriation of $1,500,000 for the work, made by the legislature of 1915, was not approved by the Governor on account of insufficient state revenue.

Notes from Field and Office

Concrete reservoir built of steel frame filled in with cement gun—New United States Coast and Geodetic Survey level rod—Test for coefficient of roughness in 22-yr. old pipe—Record forms for concrete contractors—Highway bridge loading diagrams—Concreting plant for railway bridge piers—Putting new piers under old concrete girder bridge

Building Concrete Reservoirs Without Forms

Something new in reinforced-concrete work is the construction of reservoirs and tanks by means of the cement gun without the use of forms. The steel reinforcement is first erected in the form of a self-supporting frame or cage of angles and bars, with the required amount of spiral reinforcing bars (with hooked ends) securely fastened in place. Over this frame is placed a heavy wire netting of triangular mesh, which serves chiefly to hold the concrete in place while setting, but also forms additional reinforcement.

When this entire steel structure is erected, heavy canvas or ducking is stretched over the outside, and upon this (working from the interior) a layer of concrete 1 to 1½ in. thick is applied by means of the cement gun. When this layer has set for a short time, the canvas is removed; and additional layers of dense concrete are applied from both the interior and the exterior. In this way the walls are built up to the desired thickness; and as each new layer is applied before the previous one has set, the result is a thoroughly homogeneous concrete mass. The roof is built in the same way. The surface is finished by floating.

The first structure of this type was built at Cary, Ill., in 1912. This is shown in Fig. 2. It is 18x32 ft. A larger reservoir now being built at Elmhurst, Ill., is illustrated in Fig. 1. This is 36 ft. in diameter and 22 ft. high, with a 6-in. wall, but the lower portion is below the ground. The work is being done by the Dewey Cement Gun Co., of Chicago. Harry L. Emerson, of Chicago, is consulting engineer for the Elmhurst water-works.

One advantage claimed for this type of construction is the reduced cost due to the elimination of formwork. It also produces a very dense concrete and a very tight

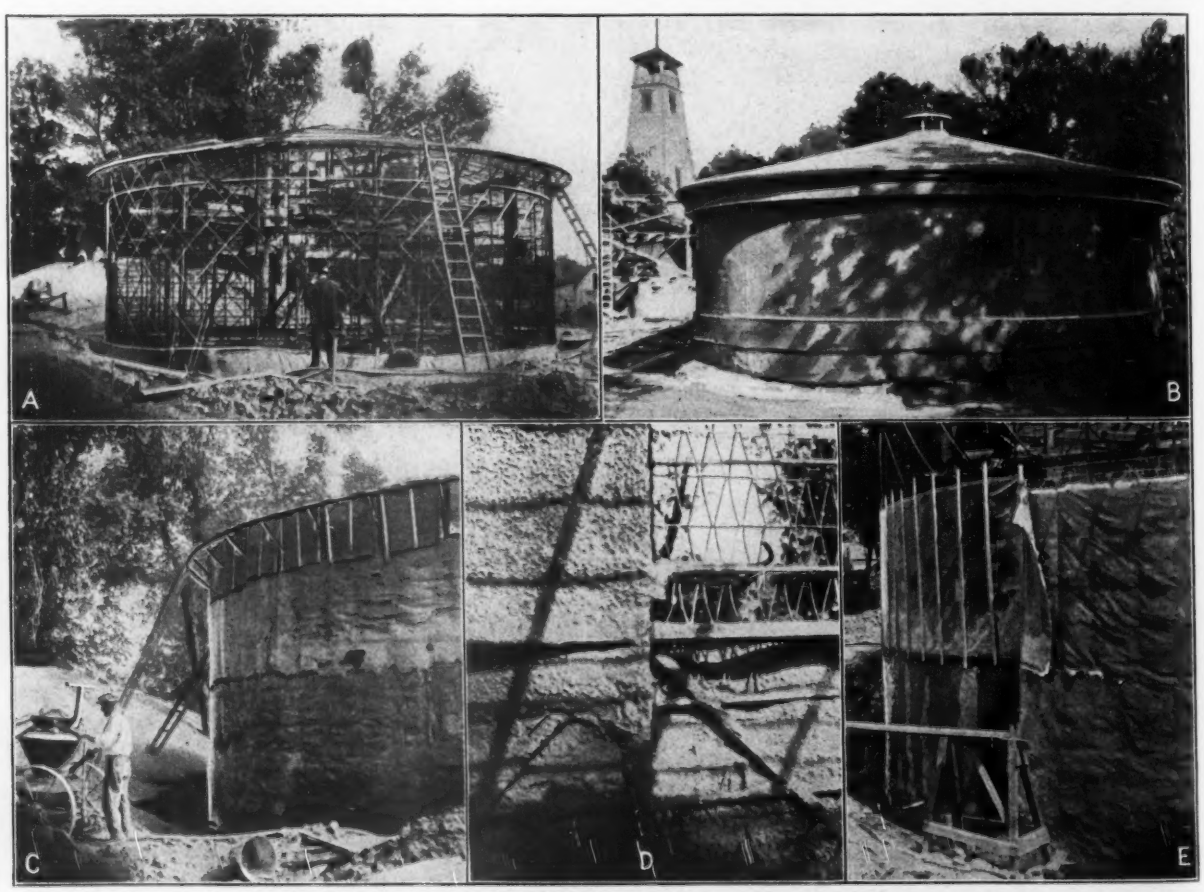


FIG. 1. BUILDING A REINFORCED-CONCRETE WATER TANK BY THE CEMENT GUN WITHOUT USING FORMS; ELMHURST, ILL.

A—The steel reinforcement erected in advance of the concreting. B—The finished structure (only the upper part is above ground). C—Exterior of rough concrete. D—Building up the concrete on the steel. E—Lower course of rough concrete, with canvas in place at the left for the next course above

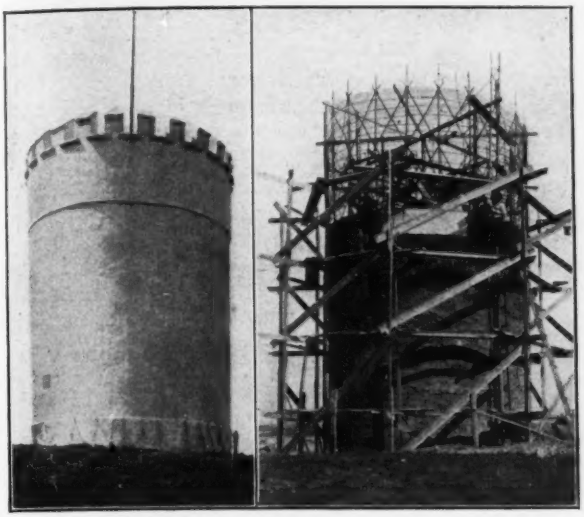


FIG. 2. REINFORCED-CONCRETE WATER TOWER BUILT WITHOUT THE USE OF FORMS, AT CARY, ILL.

structure obviating the use of special compounds in the concrete or placing a waterproof lining. One of the features of the work is that cornices of various forms can be built by an expert operator with the nozzle. These are struck off to true lines by means of a sweep and may be left in the rough, or finished to a traveled or floated surface. Successful results in the density and monolithic character of the concrete, and also in the appearance of the structure, depend considerably upon the skill of the operator handling the cement gun.

Improved Level Rod Increases Accuracy in Precise Work

By E. G. Fischer*

The level rod adopted for use in the geodetic leveling of the United States Coast and Geodetic Survey in 1900, though contributing materially toward the very great increase in miles of completed line per unit of time made since then, had certain shortcomings, which it was thought could be eliminated by placing the significant part of the graduation on a metal that, besides being structurally constant as compared with wood, should have a small temperature coefficient.

The rod now in use (which was described in Appendix 8 of the United States Coast and Geodetic Survey Report of 1899) is made of the best quality of soft white pine, submerged for eight hours in hard paraffin at a temperature of about 102° C. The centimeter graduation is placed directly upon the wood. Meter spaces of this graduation, defined by fine lines on small silver plugs, serve for occasional checking of the absolute length of the rod. The temperature coefficient was found to be uniformly 0.000004 per degree Centigrade, which is the same as that of soft white pine not treated with paraffin.

While tests, such as keeping a completed rod submerged for a day or two in water, proved that wood so treated is constant as regards moisture conditions, it was found that the rods were subject to variations in length, which can be referred only to changes in cellular or even molecular structure, incident to periods of rest, of jarring and vibrating while in the field, protracted periods of high or low temperature, etc. These variations were of sufficient magnitude to require the measuring of the meter spaces before and after each season's work, and at frequent intervals in the field.

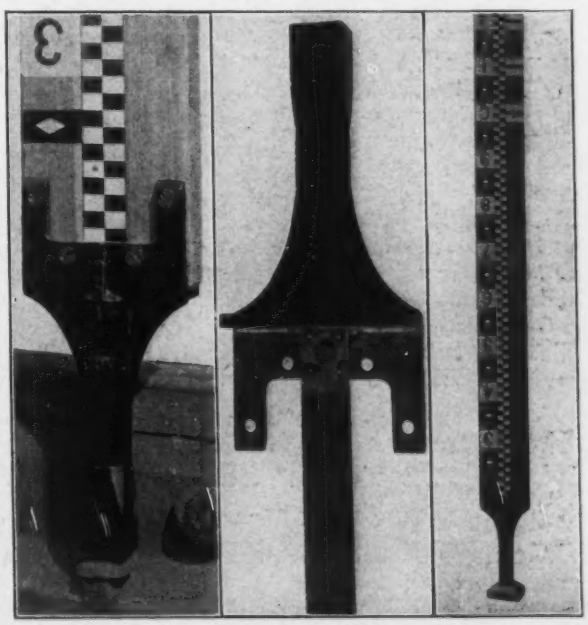
The new Coast and Geodetic Survey geodetic level rod consists substantially only of a metal foot (or shoe) to which is fastened securely a strip or band of "Gamma" steel[1] (American invar) carrying the centimeter graduation. The wooden rod, in the face of which this strip is embedded loosely and is free to move vertically, serves only to support the latter in an upright position and to carry the marks and numerals defining the decimeters and meters of the graduation of the strip. The strips are $1\frac{1}{2}$ in. wide, 0.04 in. thick and 122 in. long. Their coefficients of expansion differ slightly, but in the mean amount to 0.000001 per degree Centigrade.

As is seen from the adjoining illustration, the form of the foot differs materially from that of the present standard, which was designed to rest with its spherical lower surface either in the spherical cavity of a foot plate or pin, or, as more than 95% of the geodetic leveling is done along railways, directly upon the top of the rail. Only the greatest care could prevent errors due to instability of the foot plate. Long series of observations made with the rods supported upon the top of the rail showed errors caused by irregularities in worn rails.

The search for a more perfect support ready at hand at any place and free from these objections led to the selection of the track spike. To suit this support the lower surface of the foot was made a plane normal to the rod. The spikes being in close proximity to the rail, the brass casting holding the hardened-steel shoe was made to extend upward sufficiently to permit facing the rod in any direction, when leveling along tracks with the heaviest and therefore highest rails. For leveling away from railroads, a heavy foot pin with convex top surface is used.

*Chief, Instrument Division, United States Coast and Geodetic Survey, Washington, D. C.

[1]A nickel-steel alloy with very low thermal coefficient, made by the Midvale Steel Co.



NEW UNITED STATES COAST AND GEODETIC SURVEY PRECISE-LEVEL WORK

This method of placing the rod was tried out during the past season and was found to eliminate certain systematic errors that, no doubt, were due to habitually placing the rod differently with respect to the width of the rail when facing forward and backward.

The foot of the rod is of red metal for a height of 10 cm., with a shoe 5.5 cm. high of 1⅛-in. square steel, the lower end of which is hardened, made plane and polished. The "Gamma" steel strip is fastened to the red-metal foot by means of a stout screw, and its unvarying relation to the hardened-steel plane is secured by four steady-pins.

The wooden part of the rod is of soft, light white pine 1⅛ in. thick and 4¼ in. wide. It is not treated with paraffin, but oiled and varnished for protection against wear and decay.

The "Gamma" steel strip is held in its recess by six pairs of thin brass plates projecting slightly over its edges, but leaving it entirely free, so that changes in the wooden support cannot affect the accuracy of the centimeter graduation.

Just above the handle at the rear is embedded a small thermometer, which is shielded by a wooden sliding cover. An adjustable universal level, below the handle, enables the rodman to hold the rod vertical. The weight of the rod is 14 lb.

The bottom of the foot being of hardened steel, and the "Gamma" steel strip being secured to the foot in a manner to make them practically one piece, the index error will have to be determined only at long intervals.

Changes in Leveling Practice

Heretofore it has been the practice of the precise-leveling party to have one of the two rodmen occupy with his rod all the odd-numbered, and the other with his rod all the even-numbered, turning points, so that each rod was first shown to the instrument on the foresight; then, after the instrument was carried forward and past this rod to the next station, it was turned on its support for the backsight.

This involved at each change of instrument station the carrying forward of the rear rod past the forward rod, past the level in its new station, to the next turning point; or in other words, a distance of two turning-point intervals. By thus alternating successively the relations of the rods to the instrument accumulated errors due to uncertainties as to their lengths were reduced to a minimum.

The lengths of the new rods being known quite accurately at all times, one of them can be kept in the position to the rear and the other forward of the level for several miles at least before it is necessary to exchange them in order to avoid accumulating errors due to remaining slight uncertainties as to rod lengths. This method will save considerable time.

Roughness of Steel Pipe After 22 Years' Service

By W. L. Reynolds*

A Kutter's formula coefficient of roughness, n, of 0.00938 has been found for a 22-yr. old 16-in. (outside diameter; inside diameter, 15.5 in.) lap-welded steel pipe at Glenwood Springs, Colo. The line was built in 1888 by the Glenwood Light and Water Co., to bring water to the town for domestic uses and power. The pipe was like that now made by the National Tube Co.

A valuation of the plant was made in 1910, and it was necessary to determine both the carrying capacity and condition of this line. The following data are from a test then made to determine the coefficient of roughness.

The conditions for such an examination were exceptionally good. Three wood-stave tanks of about 90,000 gal. capacity each are direct connected to the upper end of the line. These tanks are supplied by a flume. When the test was made, the supply was cut off and the quantity of water that passed through the pipe was measured directly by recording the difference in the water surfaces. At the power house the line branches into lines supplying the wheels and the distributing mains. A gage that had recently been compared with a standard gage was used in recording pressures at the power house. The line is free from sharp curves and well below the hydraulic gradient, being under nearly 200 lb. per sq.in. pressure most of its length. A physical examination showed the pipe to be very smooth on the inside and little corroded on the outside.

TEST DATA, GLENWOOD SPRINGS STEEL-PIPE LINE

Time (Total, 25 Min.)	Water-Surface Readings, 0.0—Water-Surface with Tanks Normally Full	Water-Surface Elevations Referenced to Power-House Nozzles—0.0	Pressures at Power House, Lb.	Power-House Pressures Reduced to Head in Feet
3.14	660.00	460.00	199.6	....
3.14	1.46	458.54	154.6	....
3.16½	2.75	457.25	147.1	....
3.19	4.08	455.92	142.1	....
3.21½	5.25	454.75	141.6	....
3.24	6.35	453.65	141.1	....
3.26½	7.50	452.50	141.1	....
3.29	8.60	451.40	140.6	....
3.31½	9.69	450.31	140.6	....
3.34	10.81	449.19	140.1	....
3.36½	11.89	448.11	140.1	....
3.39	13.02	446.98	139.6	....
Averages	11.56	452.60	142.6	328.6

Volume was measured from three wood-stave tanks at head of the line. Diameter at 3.14 (time), 27.9 ft.; at 3.39, 29.4 ft.

Volume of water used, cu.ft.	22,364
Average velocity, ft. per sec.	11.38
Velocity head, ft.	2.00
Entrance head loss, ft.	3.00
Average friction head loss, ft.	128.4
Slope to overcome friction, %	0.02061
Hydraulic radius, ft.	0.3225
Coefficient C	139.6
Coefficient n	0.00938

At the head end of the line included in the length of 6,229 ft., there is 640 ft. of spiral riveted steel pipe varying in diameter from the 16-in. pipe to 24 in. at the connection with the tanks. Of course, the presence of this larger-diameter pipe would have a tendency to give a lower friction factor than the actual when the whole length was considered as of 15.5-in. inside diameter. To overcome this error, I have recomputed for a value using a total length of pipe of only 5,589 ft., or in other words I have assumed there was no friction loss in the spiral pipe. In this assumption $S = 0.02297$; $C = 132.33$; $v = 11.38$, as before; $r = 0.3225$, as before. By substituting in Kutter's formula, as before, I found a value of n of 0.00975. Therefore the correct value evidently is between 0.00938 and 0.00975. Either value is below that which is usually given for new steel pipe and perhaps only about one-half the value one would expect to find in a pipe after 22 years of service.

A check on the value obtained by this test appeared when 20.3 sec.-ft. of water was allowed to flow through the pipe and the loss as indicated by the power-house gage corresponded very closely with the computed loss.

*Glenwood Springs, Colo.

DAILY REPORT OF COST

SHEAHAN'S DAILY COST SHEET (8½x11¾)

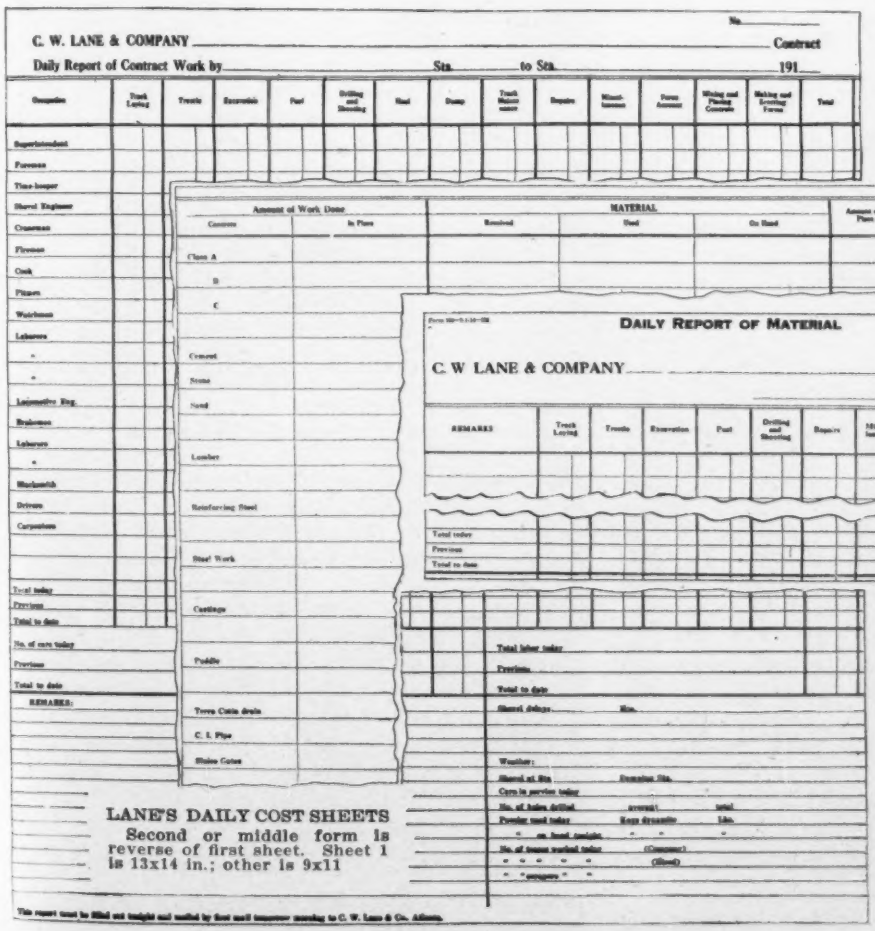
How Two Contractors Keep Daily Record of Job

The first form herewith shows how J. J. Sheahan, of Roanoke, Va., keeps tab on the cost of a contract. The figures shown are a day's record on the low-grade line of the Norfolk & Western R.R., between Burkeville and Pamplin, Va., in 1915. Such a sheet is filled out for each shift and handed to Mr. Sheahan, if he is on the work, or forwarded to him. If the foreman or superintendent has any special remarks, he writes them on the back of the sheet, upon which there is no printing or ruling.

C. W. Lane & Co., of Atlanta, Ga., uses two daily cost sheets, one each for labor and material, as shown. At foot of Sheet 1 is a reminder: "This report must be filled out tonight and mailed by first mail tomorrow to Atlanta."

Both these contractors are now working on the new second-track grading of the Southern Ry. between Central, S. C., and Cornelia, Ga., and are using here their respective methods of keeping track of costs. Lane's section is 5.5 miles long, and 8 shovels are at work on it. He uses large standard-gage locomotives to haul 12-yd. air-dump cars. Mules and 2-yd. wagons handle from one shovel (revolving). The section is 22% complete. Sheahan has a part of the Rinehart & Dennis section, and his plant comprises two shovels, six dinkeys, sixty 4-yd. dump cars, two well drills and two hand hammer drills.

Work started about the middle of May. The total yardage over the entire 50 miles is about 9,000,000. In the case of Sheahan's form, both the material and the labor accounting are on one side of the sheet, and that not only is the day's total cost worked out but that this figure is expected to be added to the previous total cost in order to obtain the total cost to date. Sheet 1 of Lane's system is for labor only. The classes of labor are listed in the first column. In the heading column are the various classes of work on which the men are employed. The reverse of this sheet is for following up the material and the third form is to be used for costs of material and works very satisfactorily.


C. W. LANE & COMPANY — Daily Report of Contract Work by ____ Sta. ____ to Sta. ____ 191__ — Contract No. ____

DAILY REPORT OF MATERIAL — C. W LANE & COMPANY — Contract — 191__

LANE'S DAILY COST SHEETS

Second or middle form is reverse of first sheet. Sheet 1 is 13x14 in.; other is 9x11

Proposed Motor-Truck Loads for Highway Bridges

By O. W. Childs*

Not many years ago bridges on rural highways were designed for a uniform live-load, and but slight attention was given to concentrated or wheel loads. Later, traction engines were assumed to be the critical loads for the floor; and still later, road rollers were used for the same purpose. Today, motor trucks are usually considered to be the bogy of the highway bridge.

Large motor trucks equal in weight the heaviest traction engines or road rollers, travel at much greater speeds and have a wider range of travel. These considerations indicate that motor trucks are the proper form of concentrated loads to specify in the design or investigation of highway bridges.

Specifications as to concentrated loads for highway bridges vary considerably. This is inconvenient and causes indefiniteness in judging the comparative capacities of structures. Traction engines, road rollers and motor trucks, with varying wheel and axle spacings and various distributions of loads to the wheels, all give different results even when the total weights of the different vehicles are the same. It is very desirable to use some standard type of concentrated load.

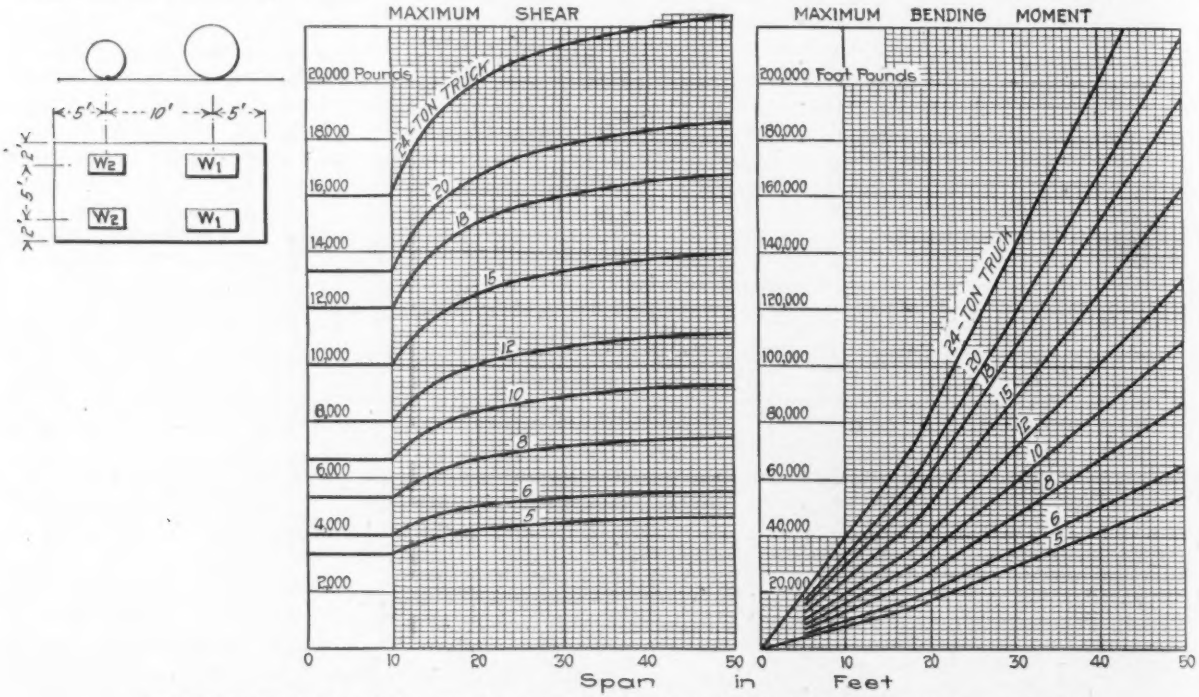
A typical truck of a varying weight serves conveniently for this common standard; when other loads, as road rollers or traction engines, must be considered, their effects may be readily expressed in terms of equivalent typical trucks.

A typical truck is herein assumed to have two axles spaced 10 ft. on centers, with the two wheels on each axle spaced 5 ft. on centers. This truck covers a space 9 ft. wide by 20 ft. long, symmetrical about the truck center. One-third of the weight is on the front axle, two-thirds on the rear axle. With this standard size and distribution the capacity may be definitely and simply expressed as the total weight in tons of the loaded truck.

The accompanying diagram of maximum shears and moments covers the range from 5-ton to 24-ton trucks, and spans up to 50 ft. The table for 10-ton trucks may be used for any weight, as the values vary in direct ratio with the weight.

Bridges on improved roads should as a rule be designed for 15-ton typical trucks, but other capacities may be necessary for special conditions. The capacity of an old bridge may be expressed as any number of tons that it may figure to carry on the wheel spacing of this typical truck.

The table and diagrams give static stresses. An addition must be made to these to provide for the effects of impact, which are more serious from swiftly moving motor trucks than from slower-moving road rollers and traction engines. In general this impact allowance should not be less than 30% of the static stress, but it should be varied to fit conditions. The nature and condition of roadway surface and the type of floor system are important factors of impact effect.



PROPOSED TYPICAL MOTOR TRUCK, AND DIAGRAMS OF MAXIMUM MOMENTS AND SHEARS

Wheel loads will usually cause maximum stresses in the floor system only, but their effect should be provided for in all members where they may cause stress in excess of that of the uniform live-load.

On rural highways only one truck need ordinarily be considered as acting on a bridge, as the roadway is narrow, in most cases. However, two trucks should be assumed as meeting and passing each other whenever the roadway is wide enough to permit this. Truck loads need not as a rule be considered as acting in combination with uniform live-load. Centrifugal force and braking effects should be taken account of where special conditions render this advisable.

*Bridge Engineer, Office of Public Roads and Rural Engineering, Washington, D. C.

MAXIMUM SHEARS AND BONDING MOMENTS FOR TYPICAL TRUCKS

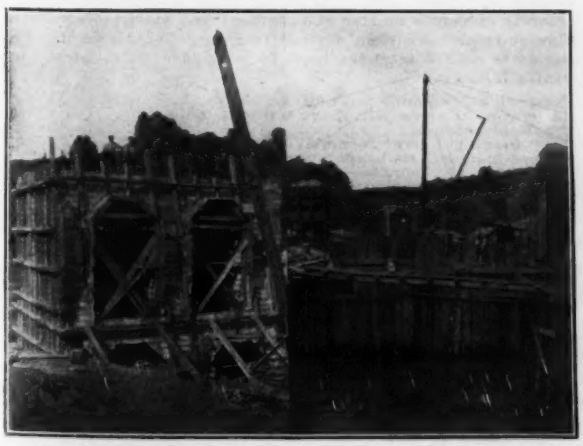
Values for two wheels on one side

Span, Ft.	10-Ton Truck Shear, Lb.	10-Ton Truck Moment, Ft.-Lb.	Span, Ft.	10-Ton Truck Shear, Lb.	10-Ton Truck Moment, Ft.-Lb.
5	6,666	8,333	28	8,810	54,325
6	6,666	10,000	29	8,852	56,791
7	6,666	11,666	30	8,890	59,259
8	6,666	13,333	31	8,926	61,740
9	6,666	15,000	32	8,958	64,201
10	6,666	16,666	33	8,990	66,675
11	6,970	18,333	34	9,018	69,150
12	7,222	20,000	35	9,048	71,627
13	7,436	21,666	36	9,074	74,005
14	7,622	23,333	37	9,100	76,584
15	7,778	25,000	38	9,124	79,094
16	7,918	26,666	39	9,146	81,546
17	8,040	28,333	40	9,168	84,048
18	8,148	30,000	41	9,188	86,511
19	8,246	32,294	42	9,208	88,993
20	8,334	34,721	43	9,226	91,477
21	8,412	37,154	44	9,244	93,965
22	8,486	39,594	45	9,260	96,451
23	8,552	42,040	46	9,276	98,946
24	8,612	44,488	47	9,292	101,424
25	8,668	46,942	48	9,306	104,012
26	8,718	49,401	49	9,320	106,400
27	8,766	51,833	50	9,334	108,889

Bridge Piers Built Quickly by Effective Plant

The second-tracked Southern Ry. will cross the Tugaloo River in South Carolina on a deck girder bridge consisting of five 100-ft. spans on two cellular concrete abutments and three concrete piers. The concrete work has been carried on with efficiency and vigor, with the result that it is expected that the bridge will be completed 10 months ahead of the contract date. Work started in April, 1915, and the contract calls for completion by September, 1917. On Oct. 1, 1916, one of the abutments (Fig. 1) was nearly finished, the excavation for the other was down 20 ft. to rock, and the pier forms were being erected. The foundation pits are dredged out by an orangepeel. The forms are very heavy and are intended for use on future contracts.

The concreting plant is designed to minimize manual labor. Crushed stone is transported to the job by a gasoline scow from a quarry located a mile or two up the river; it is unloaded by a derrick and clamshell into a bin. The bin is connected to a hopper that discharges into a ¾-yd. Ransome mixer. The concrete is conveyed to the forms in buckets on small cable cars on a track. The buckets are lifted off the cars by a derrick and dumped. The derrick then replaces the buckets on the cars and they are returned to the concrete-mixing plant.



FORMS UP FOR HOLLOW ABUTMENT OF TUGALOO BRIDGE; A PIER COFFER-DAM

The contract section of the new Southern on which the Tugaloo bridge occurs is held by Winston & Co., under whom the Shiplett Concrete Co. is building the bridge. The headquarters of G. D. Shiplett are on this job. J. O. Parsons, Assistant Engineer, supervises the work for the railway.

Close Tunnel Cross-Sectioning Widens Clearance

The Sandy Ridge tunnel on the Carolina, Clinchfield & Ohio Ry. is being lined with concrete by the pneumatic method. Before this work was started, a profile was plotted from the final cross-sections taken every 5 ft. through the tunnel. The profile showed the rock outline very accurately. Close examination of the profile brought out the fact that the interior face of the proposed lining could be shifted outward and yet adequate strength secured. Thus a large yardage of concrete was saved and a wider tunnel obtained. A small section of the profile of the tunnel excavation, from which the trimming of the lining was determined, is shown below.

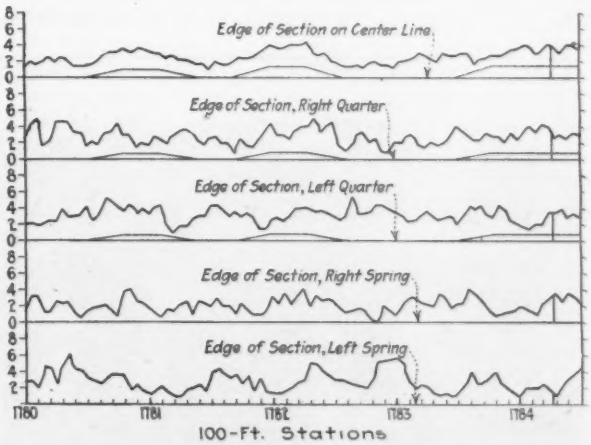


FIG. 1. PROFILE OF TUNNEL EXCAVATION, FROM WHICH TRIMMING FOR LINING WAS DETERMINED

FIG. 2. STEEL FORMS USED TO LINE SANDY RIDGE TUNNEL; NOTE ROCK

The tunnel is 7,804 ft. long and for about one-half its length is through slate and shale formations, which disintegrate upon exposure to air; the remaining half is through sandstone formation badly seamed and broken. Fig. 2 is an interior view of the tunnel, showing a 30-ft. section of steel forms in place. The finished tunnel is 19 ft. wide, 14 ft. 5 in. from subgrade to springline and has a radius of 9 ft. O. K. Morgan, Johnson City, Tenn., is Engineer in Charge.

Jacking Concrete-Slab Bridge Into Place After Failure

About a year ago an old highway bridge in Monroe County, N. Y., was replaced by a reinforced-concrete girder-and-slab bridge resting on the old abutments. The new bridge is of a novel type that has been used for several crossings in the county. Two girders at the sides rest on the longitudinal arms of a U-abutment and carry a middle floor-beam. The floor slab spans the space between the face walls of the abutment and the floor-beam. The load on the slab is thus being transmitted to the girders through the floor-beam only, as there is no reinforcement connection between slab and girder. This bridge had a span of 15 ft. and a width of 22 ft. inside the girders; the girders were 27 ft. long, extending 6 ft. beyond the face of the abutment at either end.

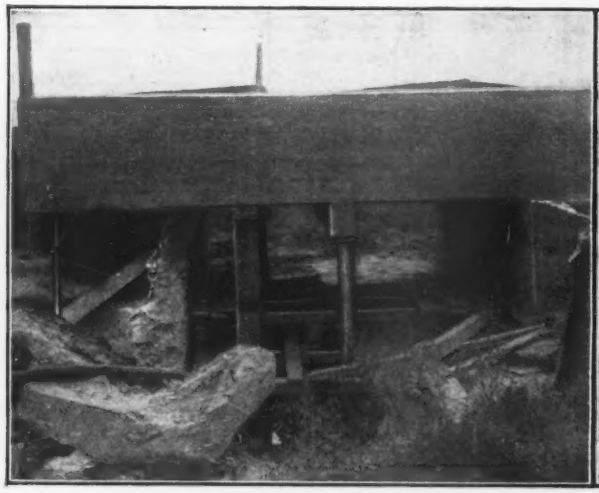


FIG. 1. MONROE COUNTY BRIDGE JACKED UP

FIG. 2. REPAIRED CONCRETE GIRDER AND SLAB

During the present summer extremely high water undermined one corner of one abutment and, washing out the earth backing there, allowed one corner of the bridge to fall. The whole superstructure, there being a construction joint between abutment and girder, toppled over as a unit toward this corner, crushing the other abutment wall on the same side as the washout and lifting off from the abutment on the opposite side of the bridge.

Examination showed that there was no damage to the bridge itself. A frame was therefore laid across the creek bed and the bridge jacked back into place, as shown in Fig. 1. Once there, it was a simple matter to concrete a new abutment side wall and to repair the face walls, so that now the bridge is doing service as shown in Fig. 2.

The design and repair work were carried out by the office of the County Engineer, J. Y. McClintock, and was in charge of George Wright, Assistant County Engineer.

FINE AGGREGATE

Laying Street Paving by Force Account is favored by the Department of Public Works of St. Paul, Minn. In the annual report of the Commissioner of Public Works, M. N. Goss, for the year 1915, it is stated that the practical effect of city force-account work during the year has been a saving of about 12% over the lowest bids submitted by street-paving contractors. The Chief Engineer of the department, O. Claussen, says: "The department is in favor of continuing to do paving by city force account for various special reasons and is splendidly equipped at the present time to lay in an efficient and economical manner 20 to 30 mi. of pavement per season." Mr. Claussen does not favor, however, the system in which the city bids against contractors for paving work, as he considers it unfair both to the department and to the contractor and predicts that it will eventually be discontinued. In comparing contractors' bids with bids made by the department for paving work, criticism has been made of the department because 3% is added to the contractors' bid to represent the cost of inspection if the work is done by contract. The extra engineering expense on contract work is certainly enough to account for 3%.

Piledriving Rules of 10 Railways—The general piledriving practice on a number of railways is included in the recent report of the Committee on Modern Methods of Driving Piles of the American Railway Bridge and Building Association. The minimum depth of penetration for piles in trestles is usually considered to be about 15 ft., but there are numberless situations where piles stopped at this depth will not sustain the load. The determination of the proper stopping place is largely a matter of judgment and experience. Formulas or rules have been devised to assist the judgment, and a summary of the methods used by a number of railroad companies follows:

Foreman's judgment without formulas	14 roads
As long as the pile will drive without damage	6 roads
Practical refusal	13 roads
"Engineering News" formula	4 roads
Special instructions based on investigation and tests	3 roads

Central of Georgia—3,000-lb. drop hammer with a 20-ft. drop, 1-in. penetration; Bucyrus No. 43, steam hammer, ¼ in. per blow.

Central of New Jersey—3,000-lb. drop hammer with a 15-ft. drop, 1-in. penetration.

Chicago & Northwestern—3,000-lb. drop hammer, 30-ft. drop, ½-in. penetration. (Above applies to foundation piles. "Engineering News" formula is also used.)

Illinois Central—3,000-lb. drop hammer, 20-ft. drop, ½-in. penetration.

Michigan Central—3,000-lb. drop hammer, 15-ft. drop, 2½ in. for last 5 blows.

New York Central Lines West—3,200-lb. drop hammer, 30-ft. drop, ½- to 1-in. penetration.

N. Y., N. H. & H.—2,000-lb. drop hammer, 25-lb. drop, 10 in. for last 10 blows.

Philadelphia & Reading—2,250-lb. drop hammer, 15-ft. drop, refusal; 3,300-lb. drop hammer, 10-ft. drop, refusal.

St. Louis & San Francisco—No. 2 Vulcan hammer, 1 in. in four to six blows.

Toledo, St. Louis & Western—Penetration of 14 ft. to 16 ft. Conditions uniform.

Editorials

Will Improvements in Water-Works Reduce Fire Rates?

Bitter complaints have been made at recent water-works conventions that after large expenditures have been made at the insistance of the Fire Underwriters and with the lure of lower water rates no reduction of rates follows. Perhaps with this in mind, Dabney H. Maury, Chicago, in a report on possible improvements to the water-works of Wilmette, Ill., concludes as follows:

> A large part of the cost of the improvements herein contemplated would be incurred for fire protection. No one will profit more directly, nor to a greater extent, from these improvements than will the fire underwriters, and they should bear their share of the cost. It is therefore recommended that these gentlemen be asked to state definitely, in advance of any expenditure for these improvements, what reduction they will make in insurance rates if the standards of service hereinbefore specified be established in Wilmette.

This seems an eminently fair question to put to the Fire Underwriters. Cities or property owners are subjected to penalties right and left for alleged, and doubtless real, deficient fire protection. What more natural than to expect that the fire-insurance rates will be lowered when a hundred thousand or a million dollars have been spent by a city on strengthening the fire service of the water-works? Perhaps rates are lowered oftener than is credited to the insurance men, but no one speaks out in meeting and says so at water-works conventions.

The fact is that the feeling most commonly exhibited at the water-works conventions is that the fire-insurance men continually cry "Give, give," to the water-works men and seem unwilling to give anything in return except what the water-works men regard as arbitrary rules with unwarranted penalties thrown in for full and running-over measure.

The outcome of Mr. Maury's suggestion to the Wilmette Village Trustees will be watched for with interest. Perhaps it will show that the Fire Underwriters are more willing to go halfway than the water-works men commonly believe.

Six Commandments of Sanitary Disposal of Sewage

Six short sentences comprise what may well be termed the Six Commandments of Sanitary Disposal of Sewage, recently submitted as a proposed code by the Committee on Sanitary Control of Waterways of the Sanitary Section of the American Public Health Association. The six rules, together with the committee's explanatory notes, are printed elsewhere in this issue. They deserve the attention of every reader of this journal.

Rarely does a code so clearly crystallize the strong belief of the vast majority of engineers working in an important field as do these six commandments. We therefore expect from engineers no material objection to the code's requirements. That there may be considerable protests to the proposed code we have no doubt, but for the most part they will be in the nature of outbursts from persons with more zeal than knowledge.

It is unfortunate, but true, that popular zeal for the protection of natural waters from pollution—zeal that was long in being aroused—has far outrun knowledge of the real effects of discharging sewage into natural waters, of what is practicable in the way of sewage treatment, and of the relationships of sewage treatment and of water treatment to stream pollution and to each other. The fundamental principles succinctly laid down by the committee in its report and the well-considered notes to each of the six sections of the code provide needful means of popular education on these subjects.

The code and its prefatory fundamental principles, it is hoped, will soon be brought to the attention of the general membership and of the other readers of the "Journal" of the American Public Health Association. So far, good; but this is not enough. The whole report deserves much wider circulation. The cause of public sanitation and sound public economics would be well served and at the same time the American Public Health Association would be given helpful and deserved publicity if the entire report were reprinted and widely distributed for publication and comment in the daily and weekly press of the country.

Road Surfacing Is Not Road Building

Since the beginning of the extensive use of pavements on rural highways the term "road building" seems to have become confused with pavement construction, or "road surfacing." There is a difference. Road building is a much broader term; it includes more than a superficial improvement, in a literal as well as in a figurative sense. The Romans were road builders, and their roads have endured, while modern road surfacings have worn out in a very few years.

The permanent character of the Roman roads was of course attained at too high a cost to be emulated by modern highway engineers, yet there are certain permanent features in everyday modern road building which ought not to be neglected; and there seems to be plenty of foundation for believing that they are neglected, and that a little of the effort and talent being directed to find an ideal surfacing material might well be addressed to some of these simple engineering essentials.

What some of these essentials are H. E. Bilger, Road Engineer of the Illinois State Highway Commission, points out in another part of this issue. Little of this matter is new. In fact, most of it is rather hackneyed, yet it seems that constant reiteration of these principles is necessary to insure consideration.

Tons of literature have gone forth on the necessity of proper drainage of roads, but let this significant sentence from Mr. Bilger's article sink in: "In order to maintain satisfactorily a road surface in practically level country on other than a sandy soil, it is generally abso-

lutely necessary to forestall capillarity by elevating the roadbed above the level of the adjacent fields."

Obvious and platitudinous as this may seem, there is no elementary principle of road building that is oftener violated. Anyone who has traveled over some of the infamous roads of Kansas, Nebraska, Missouri and some of the other agricultural prairie states can vouch for the fact that a great part, if not all, of their troubles is caused by the fact that the roadbeds are troughs in cultivated fields. No amount or kind of surfacing on such roads will cure their real weakness.

It is strange that a practice so universally followed in railway grading should be so often ignored in the parallel practice of highway grading, although behind this neglect of course is the desire to save expense. To raise a 21-ft. roadbed 1 ft. above the level of the surrounding land requires about 4,500 cu.yd. of fill per mile, making an allowance of 10% for shrinkage. Made with wheel scrapers from roadside borrow pits at 20c. per cu.yd., this extra cost would be $900 per mi.—a liberal estimate.

Suppose we go a little farther than Mr. Bilger and suggest a nearer approach to railway practice—an 18-in. fill—making the additional cost not over $1,500 per mi., or 10% of the probable pavement cost, which is not an extravagant expense when the permanency of a $15,000-per-mi. investment is under consideration.

American Concrete Institute Placed on a Sound Basis

Those connected with the concrete industry who are aware of the difficulties through which the American Concrete Institute has traveled in the recent past will be pleased to learn, in connection with the announcement of the annual convention to be held next February, that the Institute is now in a financially sound condition. Back numbers of the Institute's Proceedings, which have long been withheld from publication for lack of funds, are shortly to be printed and placed in the hands of the members. The delayed volumes Nos. 9 and 10, containing the Proceedings of the conventions of 1913 and 1914, will be printed and distributed to the members before the end of the year. The Proceedings of the convention of 1915 were printed during that year in the form of a monthly journal. Vol. 12, which contains the Proceedings of the convention of last February, has been printed, and is now in the hands of the binders. The cash necessary for publishing all these volumes is already in hand.

The concrete industry owes a debt to the officers of the Institute, headed by President Leonard C. Wason, for the energetic manner in which they have discharged the responsibilities laid upon them. There can be no dissent from the proposition that the Institute has in the past done a very important work for the sound, technical development of the concrete industry. It is the record of that past work, doubtless, which has made possible the campaign that has placed the Institute on a sound financial basis.

The rapid growth of the industry ought to make possible a large accession of membership to the Institute now that it is firmly established as a successful going concern. The President announces that fourteen committees are at work upon subjects in connection with the concrete industry in order to have their reports ready for presentation at the February convention. Under these conditions many new members should join the Institute at the present time so that they may be either able to attend and take part in the February meeting or entitled to receive the report of its proceedings.

Successful Train-Braking with Electric Motors

Saving the power generated when a train runs down a grade and utilizing that power to help the train up another grade has been a dream of inventors ever since the early days of the railway era. The invention of the electric motor and its application to railway service seemed to open the way for the possible realization of this dream, since the electric motor is capable of acting both as a motor and as a generator.

In the November "General Electric Review," J. J. Linebaugh, of the General Electric Co.'s railway and traction engineering department, reviews the progress of regenerative electric braking. The problem may seem easy to the engineer unfamiliar with electrical design; but while a great many different combinations of fields, armatures, motors and control have been invented to make possible regenerative electric braking, none of these combinations have until recently been found practicable with the direct-current railway motor.

With the three-phase alternating current the induction motor used can very readily be adapted to electric braking on steep grade lines, and such a system has been in operation for many years on some of the Alpine lines of the Italian state railways. The three-phase system has been strongly urged as preferable to the direct-current system in the electrification of steam railways on account of this advantage.

The principal installations of electric braking in this country on alternating-current installations have been in connection with the electrification of the western division of the Norfolk & Western Ry. and on the electrified section of the Great Northern Ry. through the Cascades tunnel, the former on the split-phase and the latter on the three-phase system.

One of the most notable features in connection with the Chicago, Milwaukee & St. Paul's electrification of its main line through Montana and Idaho is that electric braking has been adopted and made successful with the 3,000-volt direct-current system. In fact, the braking system is claimed to be superior to that used with the induction motor on alternating-current railways. With this latter type of motor as used on the Norfolk & Western Ry., for example, electric braking is only operative at one uniform speed. In other words, when the train is running down grade and the motors are acting as generators, the train is held to one uniform speed regardless of changes in the profile and alignment.

In practical railway operation, however, it is often desirable to make large variations in speed when running down grade, and the direct-current motor employed as a generator permits this. The control adopted on the St. Paul lines allows the motors when used as generators to be operated either in series or parallel. With the flexibility thus secured it is possible to do effective electric braking over a wide range of speeds. Mr. Linebaugh submits cards of braking effect from a freight locomotive at speeds of 8 to 35 mi. per hr. and from a passenger locomotive at speeds of 15 to 65 mi. per hr.

The success of this electric-braking system is claimed to be due to the commutating pole motor used, which, with its thorough internal ventilation, can stand without injury the heavy overloads due to fluctuating line voltage, and also to the simple form of control that was worked out by the General Electric engineers just before the St. Paul contract was let.

It is evident that if regeneration is used, all the apparatus on the locomotives and in the substations, etc., must be capable of operating inverted. Not only must the motors on the locomotives act as generators, but the generating equipment in the substations must invert; and if motor-generators sets are used, the direct-current generators must operate as motors and the synchronous motors as generators. They must be capable of doing this instantly and as often as required without affecting reliability or successful operation in any way.

The electrical machinery installed on the St. Paul system has been designed to meet these requirements. When a train is descending a heavy grade and feeding current back into the line, that current is utilized in the propulsion of trains on other portions of the line. Under the rare conditions when more power is being returned to the line than is required by other trains, the surplus power is available for other demands over the wide network of lines fed from the power houses that supply current to the railway. This returned power is in fact credited to the railway company.

On the electric-braking system used on the Norfolk & Western Ry. the power house supplies current to the railway alone, and it was necessary to install large water rheostats to turn excess current into heat, when the railway is returning energy to the power house instead of drawing from it.

The profile of the electrically operated line of the St. Paul shows grades of 2% as a maximum, while there are long stretches of 1.7% grades, and three mountain summits are crossed at elevations of 5,678 ft., 6,350 ft. and 4,200 ft. The electric motors are used as brakes only in descending the steeper grades. On a grade of less than ½% there is little or no surplus of power over that required to overcome the train resistance. On grades of 0.6% one-half the locomotive is used to generate power. It will be understood that the air brakes are regularly used for stopping and holding trains and as an auxiliary for regulating speed when the electric motors are acting as generators.

The great advantage of the electric-braking system in securing the safe handling of trains on long heavy grades will be obvious to every railway man who is familiar with the precautions necessary for the safety of trains on such grades with air brakes alone. The St. Paul is sending trains of 3,000 to 3,500 tons' weight behind the tender down these mountain grades, often with a single locomotive.

It will be readily understood that, with electric braking, the train is bunched against the locomotive in descending a grade, and the common difficulty with broken draft gear due to unequal action of air brakes, varying profile, etc., is eliminated. Of equal importance is the elimination of overheated wheels and brake shoes, with the resultant liability to accident.

These practical advantages in the operation of trains on mountain grades are probably more important than the saving in power by the use of the motors as generators although the power saving is considerable. Mr. Linebaugh states that in descending a long 2% grade a train will return to the line about 60% of the current that would be required to haul it up the same grade. In actual service the power saving varies of course with the grades operated over. On eastbound St. Paul trains between Deer Lodge and Three Forks the power returned to the line is about 25% of the power drawn from it. On trains in the reverse direction over the same division 7% is the proportion returned. On the entire 220 mi. between Deer Lodge and Harlowton the power returned by all trains both ways is about 14% of the total current consumption.

The foregoing summary of what has been actually accomplished with electric braking presents facts of interest and importance to every railway engineer. Numerous problems of economic railway location may require revision in the light of these facts. Just as the application of mechanical power to road vehicles has radically revised standards of road construction, so the introduction of electric motive power and regenerative braking may cause a new answer to be given to old problems.

To take a concrete example, engineers are generally familiar with the extensive line revision carried out by the Delaware, Lackawanna & Western R.R. a few years ago, west of Scranton, Penn., at a cost of about $12,000,000, by which 40 mi. of new railway were built to save 327 ft. of total rise and fall with a reduction also of maximum grades and of curvature. It is at least within the possibilities that the introduction of electric motive power over this division might have been a much more economical solution of the problem, since it would also have improved operating conditions on the long and heavy grade that still remains on the line west of Scranton.

This is only a typical example of the problems in railway location and relocation where the engineer will need hereafter to take into consideration the fact that the use of electric motive power with electric braking has materially reduced the difficulties and losses in the operation of railways with heavy grades.

How To Use Your Old Canals

For many years a number of American cities have suffered from the nuisance of abandoned canals in their midsts—canals that not only were obnoxious as open sewers, but were worse than useless because their boatless waters took up useful ground in the busy areas of the town. Now several of these communities are finding, or hope to find, a use for these deep water-filled cuts by draining the canal and using the bed for rapid transit or railway lines. In Cincinnati the proposed interurban entrance is on the Miami and Erie Canal there; in Syracuse the old Erie Canal bed will certainly in time become the route of the New York Central through the city to replace the tracks which now are the most disgraceful example of municipal disfigurement in the country; in Rochester the Erie Canal will soon be turned into new channels, permitting the use of its bed for a street railway route. The combined advantages of ridding the city of a nuisance and of obtaining ready made a depressed railway terminal or rapid transit route are of such commanding importance as to recommend the procedure to any city now saddled with an unused canal.

Letters to the Editor

Truck Springs Save Pavements?

Sir—Regarding the editorial on p. 710 of *Engineering News*, Oct. 12, on pavement destruction by motor trucks, it might be interesting to note that agitation looking toward the elimination of springless trucks has been in progress for some time in San Francisco, Calif.

The effect of the efficiency of springs on the impact of heavy loads can be seen very easily, as also the probability of objections to springs that might tend to throw the load out of the container. T. H. CLAUSSEN.

Stanford University, Oct. 16, 1916.

[The particular trucks referred to in the editorial mentioned, and in the article on page 691 of the same issue, have springs; but springs to carry 18-ton loads are about as stiff and useless as locomotive springs to save wheel shocks.—Editor.]

Gravel Covers on Concrete Bridge Floors

Sir—In the issues of *Engineering News* for Sept. 7 and Oct. 12 are letters from C. E. V. and C. E. Nagel respectively, regarding the use of gravel covers on concrete bridge floors. Mr. Nagel makes four categorical statements, which, with your permission, I will reply to *seriatim:*

1. The gravel covering is less expensive than the extra thickness of concrete floor and weighs no more.

The question of the comparative cost of gravel as against that of an extra thickness of concrete is one governed entirely by the cost of each material at the site, which of course varies greatly according to locality; and also with the volume and nature of traffic, by which the required extra thickness of the concrete wearing surface should be determined.

Unless the gravel contains a suitable sand-clay binder and is laid at least 5 or 6 in. thick, experience shows the gravel cover difficult to maintain. The extra weight of gravel cover as against the extra thickness of concrete required calls for a heavier design in the bridge, when load requirements in the design receive the care they should.

2. Troweling the top of the slab is not necessary when a gravel covering is to be placed, making a further saving in cost.

Granted. Neither is troweling required—in fact, it is a detriment—when an asphalt or tar composition is used.

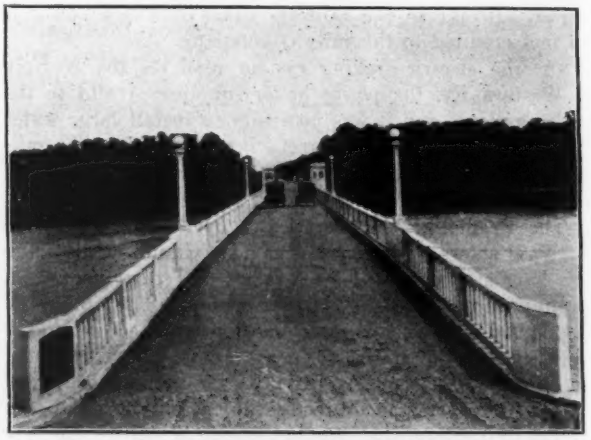
3. Chuck holes at the end of the bridge are avoided where the surface material on the bridge is the same as that on the road.

This contention is justified to a certain extent. If the approach fill is well consolidated, this trouble is minimized; but even then, and with a continuous surface of gravel, chuck holes will eventually develop. It is this feature that has almost entirely governed the writer in using a gravel cover continuous with a gravel road on *short* bridges, as the proportional area of trouble arising from chuck holes relative to floor area on short bridges is great. In short, the continuity of the surfacing material does minimize this trouble, especially with care in construction of fills.

4. The gravel covering is of great assistance in properly curing the concrete slab.

With this I cannot agree. The period of curing is most important during the first seven to ten days after pouring. Generally speaking, concrete bridge floors are poured some little time before the approach fill is put in; consequently, the placing of a gravel cover is not practicable at such a time. A good curing blanket can be obtained with either earth or sand kept damp, or where the floor is on a level grade, a clay dam at each end and a few inches of water maintained over the slab will be found to give excellent results.

The writer has experienced considerable trouble caused by the gravel covers washing and raveling from effects of rain. A bridge deck thus covered is also difficult to drain without impairing the condition of the surfacing. The writer's experience points favorably to the use of a tar or asphalt composition placed as thinly as possible, with a good blanket of torpedo sand cast on before the composition cools and sets.



TAR-CARPETED FLOOR OF CALCASIEU RIVER BRIDGE, NEAR LAKE CHARLES, LOUISIANA

Traffic over the concrete deck should first be continued until the aggregate is exposed and all laitance removed. Good results have been obtained from a treatment of 0.3 gal. of tar to the square yard. Great care should be taken to see that the concrete surface is quite clean and dry and that the sand blanket is maintained until all signs of bleeding have passed.

The writer had a number of concrete bridges treated about 14 months ago in this manner, and despite a heavy traffic the surfacing is still in first-class condition and probably will not require resurfacing for several months yet. The Calcasieu River bridge near Lake Charles, recently completed from the writer's plans and specifications and under his general supervision, received a

wearing-surface treatment of 0.3 gal. of tar per square yard, with a torpedo-sand blanket, at a cost of 4⅞c. per sq.yd. for approximately 3,200 sq.yd. The treatment was put on by day labor.

The use of a plain concrete wearing surface on bridge floors subjected to light traffic has proved quite satisfactory.

The writer's conclusions stated above have been arrived at from a number of years' experience in the design and construction of concrete highway bridges in many localities under very varied conditions.

F. H. FRANKLAND.

Lake Charles, La., Oct. 15, 1916.

Busiest City Thoroughfare?

Sir—In *Engineering News* of Oct. 19, 1916, p. 761, L. S. Brodd quotes a police count published by the Chicago *Tribune*, giving extremes of traffic on Chicago streets, as follows:

> On State St., between Madison and Washington St., 11,674 vehicles between the hours of 8 a.m. and 6 p.m. of July 19, 1915, or 1,167 vehicles per hour. On Rush St. bridge, 13,600 vehicles between 9 a.m. and 6 p.m. of July 13, 1915, or 1,511 vehicles per hour.

In *Engineering News* of Sept. 28, 1916, p. 597, it is stated that on Fifth Ave. in New York, at 42d St. 1,149 vehicles were counted proceeding south in the hour between 3:30 and 4:30 p.m.

These figures are represented as maxima for the world. I find, however, in a published report of the Traffic Branch of the London Board of Trade in 1913 that in Piccadilly, the most heavily traveled of all streets in London, 31,335 vehicles passed the Hotel Ritz in 12 hr. This is 2,611 vehicles per hour. The heaviest bridge traffic in London is on London Bridge. The figures are 17,250 vehicles in 12 hr., or 1,438 vehicles per hour. These are much higher intensities of traffic than the figures for either Chicago or New York.

CHARLES L. STROBEL,
President, Strobel Steel Construction Co.

Chicago, Ill., Oct. 24, 1916.

[A traffic census at Fifth Ave. and 42d St., New York, on Oct. 18 by the Fifth Avenue Association showed 17,151 vehicles moving north and south between the hours of 8 a.m. and 6 p.m. This gives an average for the 10 hr. of 1,715 vehicles per hour, which is much more than the Chicago count. Moreover, it must be remembered that 42d St. is almost equally congested, and that every effort is being made to divert traffic from Fifth Ave.—Editor.]

Combination of Plane Table and Transit for Topography

Sir—In your issue of Sept. 14, 1916, p. 488, is described a system of surveying using a combination of plane table and transit. The undersigned some years ago developed this system almost exactly as described and explained its use in the textbook entitled "Military Topography," written in 1910. The following extracts are from this book, p. 166:

> PLANE TABLE AND STADIA SURVEY, WITH TRANSIT USED TO READ STADIA
>
> 163. The transit may be used for reading the stadia and vertical angle when the plane-table alidade has no telescope, by having the transit and plane table set up side by side at each instrument station and about 4 ft. apart to allow the observer to pass between them. The observer sights each rod position through the sighting slits, draws the radiating line, and while he is doing this the recorder is reading the stadia and vertical angle on the rod. The recorder then reads from the computer the true distance and elevation of the point sighted to the observer, who plots them as in the method first described, for plane table with telescope. . . . It is evident that the work can be done much more rapidly by this method than with the plane table alone, because of the more equal division of the labor between observer and recorder. The observer can devote more time and study to the ground and its contouring, and the assistant to estimating distances, slopes and elevations. This method also has the advantage that stadia readings and vertical angles are more rapidly and accurately read with a transit than with a plane-table alidade because of the difficulty of maintaining the large drawing surface of the plane table in sufficiently exact level to give accurate vertical angles. The telescope alidade is heavy and awkward to use and carry from one station to another. In taking readings with it the observer is very liable to lean down on the plane table so that his breast rests against the board. If this occurs his breathing causes the board to move up and down, deranging the reading on the rod. Therefore, always use the transit and plane table where a transit is available.

In reasonably open country this system is in my opinion the most rapid and economical method of taking topography as well as the most accurate of all the available methods usually employed. I was glad to observe that the writer used the plane table as a *plane table* and not as a drawing board for protracting the angle readings taken with the transit.

C. O. SHERRILL,
Major, Corps of Engineers, U. S. A.

Ancon, C. Z., Sept. 28, 1916.

Wood-Block Expansion Joints in Concrete Pavements

Sir—I note with interest what the Michigan State Highway Department concludes about eliminating expansion joints in concrete pavements (see *Engineering News*, Sept. 28, p. 609).

Our experience with the type of expansion joint in use here, covering a period of five years on streets and roads of varying traffic, has been so satisfactory that it should be of interest to your readers.

We use creosoted wood blocks, 16-lb. treatment, sawed out of commercial-size 2-in. lumber in length equal to the depth of the pavement. These 2-in. blocks are placed in a continuous row across the pavement with the tops flush with the top of the pavement and with the grain of the wood vertical.

To prevent the blocks from being hammered down under traffic we place a strip of concrete 6 in. wide and 4 in. deep under the row of blocks. To place them evenly a lath is tacked on each side and nailed to short stakes driven into the ground.

We require the day's work to stop at a joint and space joints about 30 ft. apart. Transverse cracks are practically unknown, and the joints have not needed any attention; even those five years old are in perfect condition. In hot, dry weather the wood shrinks as the concrete expands, and it swells again by absorption as the concrete contracts in wet, cooler seasons.

The cost here is about 10c. per lin.ft. of joint, or about 3½c. per sq.yd. of pavement.

P. L. BROCKWAY,
Assistant City Engineer.

Wichita, Kan., Oct. 13, 1916.

[It should be added that the theory proposed above, that the wood swells in wet weather while the concrete contracts, is contrary to the now generally accepted fact that concrete expansion is caused fully as much by the absorption of moisture as by heat.—Editor.]

The Battle Over the Miami Flood-Prevention Plans

By H. M. Chittenden*

SYNOPSIS—The most important work for river regulation and flood control ever undertaken in the United States (aside from the levees on the Mississippi and Sacramento Rivers) is that now projected for the control of the Miami River in central Ohio. For a month past a court has been hearing the testimony of distinguished engineers on the question whether this work should proceed. At the request of the editors of "Engineering News," Gen. H. M. Chittenden, of Seattle, one of the eminent engineers summoned to testify, has prepared the following summary of the evidence thus far presented for and against the project.

The Conservancy Court of the Miami Conservancy District has been holding a public hearing at Dayton, Ohio, for a month on objections to the official plan for flood protection which was adopted by the directors of the district on May 10, 1916. The hearing has been full of interest throughout and has led to some developments of value to the engineering profession. The following is a résumé of events leading up to the hearing and of the chief features of the Ohio conservency law:

CHRONOLOGY OF PROJECT

1913, Mar. 22-27—Occurrence of the greatest flood in the recorded history of the Miami Valley.

1913, May 27—Fund of $2,000,000 subscribed with which to carry on work of formulating a plan of flood relief for the valley. Fund administered through a Flood Prevention Committee of the Citizens' Relief Commission, an incorporated body.

1913, May 5—Morgan Engineering Co., Memphis, Tenn., engaged to investigate the physical problem and present a scheme for its solution.

1913, Oct. 3-4—Preliminary report by Morgan Engineering Co. on different projects for flood protection, and definite pronouncement in favor of retarding basins.

1914, Feb. 17—Conservancy law of Ohio enacted. (See outline given below.)

1914, Feb. 18—Petition asking for creation of Miami Conservancy District filed in Court of Common Pleas, Montgomery County.

1914, Mar. 26—Report of Special Board of Consulting Engineers. The preliminary hearing on above petition was to have been held Mar. 20, 1914. To assist in presenting its case, a board of engineers, consisting of men who had some special familiarity with problems of this character, was summoned to Dayton. Legal obstacles having indefinitely postponed the hearing, the board, before leaving Dayton, submitted a report upon the plan of flood prevention as so far developed.

1915, June 3—Court of Appeals upheld constitutionality of law. Case appealed to Supreme Court and after a great deal of litigation.

1915, June 4—Supreme Court definitely upheld entire law.

1915, Apr. 21—Attempts to alter or annul conservancy law by legislative action finally defeated.

1915, June 24—District finally fixed at its present size with nine judges constituting Conservancy Court.

1915, June 28—Miami Conservancy District established; E. A. Deeds, of Dayton, H. M. Allen, of Troy, and G. S. Rentschler, of Hamilton, made Board of Directors.

1915, July 7—A. E. Morgan appointed Chief Engineer.

1915, Aug. 4—Board of Appraisers appointed in accordance with Section 26, conservancy law.

1916, Mar. 1—Report of Chief Engineer submitting plan for flood prevention filed with Board of Directors.

1916, Apr. 10 to May 10—Hearings by Board of Directors on plan of Chief Engineer.

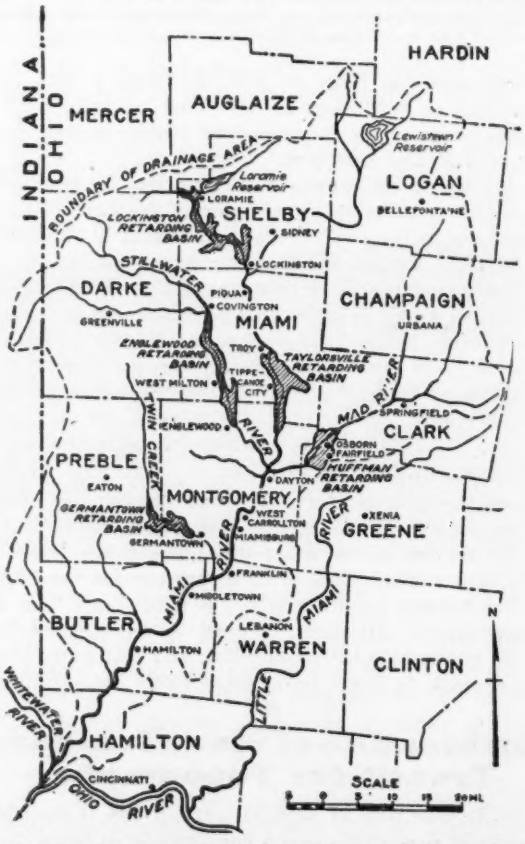
1916, May 10—Plan of Chief Engineer adopted as official plan of district. Objections having been filed with the Conservancy Court, a date was fixed by the court for a public hearing thereon.

1916, Oct. 3—Public hearing began.

CONSERVANCY LAW OF OHIO

Existing legal machinery being inadequate for the proper handling of a problem like that of controlling the floods of the Miami, a law was passed by the state legislature authorizing the creation of conservancy districts for the purpose. The law, which was drafted under Mr. Morgan's supervision, is very comprehensive. The order of procedure under it, down to the stage which the Miami project has now reached, is given in outline below:

1. Organization of districts is initiated by petition under specified restrictions to the Court of Common Pleas of some



MAP OF MIAMI RIVER-DRAINAGE AREA, SHOWING PROPOSED DETENTION BASINS FOR FLOOD CONTROL

county in the district affected. If the territory shall extend into more than one county, the court shall consist, for the purposes of the act, of the judges of the several counties and shall sit in the courthouse of the county where the original petition is filed.

2. Before a district can be organized, the court must hold a public hearing at which all objections may be heard. Upon favorable finding by the court the organization of the district is decreed. The decision of the court is subject to the usual judicial appeals.

3. Within 30 days after the decree incorporating the district the court shall appoint a board of three directors, with specified qualifications, terms of service, etc.

*Brigadier General, Corps of Engineers, U. S. A., retired.

4. Among other details of organization the Board of Directors is required to appoint a Chief Engineer.

5. The Board of Directors must forthwith cause to be prepared a plan for carrying out the purposes of the district. Before the plan is adopted the board must hold a public hearing. After the hearing the board may adopt the plan, which then becomes the official plan of the district, subject to modification as provided in the next section.

6. If objections to the plan are filed with the Conservancy Court within a specified time, the court must itself hold a public hearing and pass upon the question raised. In case of rejection of the plan a new plan must be prepared. In case of reference back for modification the hearing shall be continued. In any case it is contemplated that a plan will be finally adopted. It is this hearing by the Conservancy Court which is now in progress.

The Official Plan

Although quite a complete description of the plan has already appeared in *Engineering News* (Apr. 6, 1916), the following condensed summary of it, together with the accompanying sketch map, will contribute to the convenience of readers.

The plan is a very carefully worked-out combination of reservoir control and of channel improvement. By neither method alone could the problem be solved. The cost of protection against a flood like that of 1913 by channel improvement exclusively would have been absolutely prohibitory. On the other hand, reservoir sites were not available that would control the discharge of such a flood so that no channel improvement through the cities and towns would be necessary. But it *was* found possible to secure sufficient storage to reduce the flood discharge to a point where channel improvement was entirely practicable and within a cost for the whole scheme which bore a fair relation to the benefits to be received. The following are the chief features of the final plan:

1. The plan provides for a storm 40% in excess of that of 1913; that is, it is planned on this basis, but as actually developed it will do much more than that. The dams will be perfectly secure against a flood more than twice as great as that of 1913, while the freeboard on the levees through the cities is 3 ft. or more above the estimated flood plane for a flood 40% greater than that of 1913.

2. The reservoir control is to be accomplished by five retarding basins. For several reasons the possible scheme of storing the water for industrial or other use was rejected. The reservoirs will be for flood prevention only. Their functioning will be purely automatic. The outlets are to have no gates, but to remain at all times wide open. The ordinary discharge passes freely. Only in time of freshet will there be any accumulation, and such as does occur, even in great floods, will all run out in a few days. As these floods nearly always occur in the winter or early spring, the basins will be free of water during the crop season and practically as useful for cultivation as at present.

2. There are to be five of these basins. Their names and locations are shown on the map. One is above Piqua; four are above Dayton; all are above Hamilton. With almost incredible pains the problem of balancing these reservoirs so as to prevent undue flooding in certain cases, yet so to control the outflow of all as to insure the desired restriction of flood discharge, was eventually worked out. The more important data pertaining to the system are given in the accompanying table.

3. Through the cities or villages of Piqua, Troy, Dayton, West Carrollton, Miamisburg, Franklin, Middletown and Hamilton, channel improvements will be made to carry a flood flow, as regulated by the basins, 40% in excess of 1913. These improvements will vary to suit the particular case and will include channel rectification, dredging, levees, slope paving, etc.

4. The execution of the plan will involve considerable relocation of roads, railroads and other facilities, public and private, and the acquisition, either by easement or fee, of considerable areas of land.

DATA PERTAINING TO DAMS FOR FLOOD CONTROL OF MIAMI RIVER

	Germantown Dam	Englewood Dam	Lockington Dam	Taylorsville Dam	Huffman Dam
Average elevation bed of river	723	768.0	876	759	777
Average elevation valley floor	730	782.0	885	770	785
Elevation of spillway crest	815	876.0	938	818	835
Elevation of top of dam	830	892.5	954	837	850
Maximum elevation of water surface for a flood like 1913	805	862.0	936	815	830
Same for 40% greater flood	815	876.0	940	820	835
Same for twice as great a flood	824	887.0	948	832	846
Area submerged in a flood like 1913, acres	2,950	6,350.0	3,600	9,650	7,300
Maximum amount of water stored in a flood like 1913, acre-feet	73,000	209,000.0	63,000	152,000	124,000
Length of crest of dam, ft.	1,210	4,660.0	*6,400	*2,980	*3,340
Thickness of dam at average elevation of valley floor, ft.	650	740.0	410	390	380
Length of outlet conduit, ft.	520	725.0	45	40	40
Number of conduits	2	2.0	2	4	3
Total sectional area of outlet conduits, sq.ft.	182	21.40	158	1,116	705
Maximum discharge of conduits in 1913 flood, sec.-ft.	9,340	11,000.0	8,630	51,300	32,600
Same for 40% greater flood	10,000	12,000.0	9,000	55,000	35,000
Same for twice as great a flood	16,100	26,500.0	18,100	79,400	48,400
Length of spillway crest, ft.	55	100.0	72	132	100

*Including spillway crest.

Such, in briefest summary, is the official plan. The object of the present hearing by the Conservancy Court at Dayton is to consider objections that have been raised to the plan.

Routine of the Hearing

The Conservancy Court consists of nine judges, one each from the counties of Hamilton, Butler, Warren, Preble, Montgomery, Greene, Clark, Miami and Shelby, all of which counties embrace lands within the limits of the district. The judge from Montgomery County, in which the original petition was filed, is presiding judge of the Conservancy Court. Questions before the court are decided by majority vote of those present. The court sits on Tuesday, Wednesday, Thursday and Friday of each week, from 10 to 12:30 a.m. and from 1:30 to 4:30 p.m. It has very properly taken the ground that the nature of the inquiry before it requires some relaxation of the rules of evidence as applied in a strictly judicial trial, and witnesses are given all reasonable latitude in answering questions and explaining their views.

The first session of the court was taken up with organization and with statements by counsel for the district and for the opposition of the general plan of flood protection, on the one hand, and the basis of opposition to it, on the other. The next three sessions were consumed in viewing the sites of the several dams, so that the hearing did not commence in earnest until Oct. 10.

The Conservancy District is represented at the hearing by its legal staff, including Attorneys Orrin B. Brown and John A. McMahon, of Dayton, and John Galvin, of Cincinnati, who are taking active parts in the conduct of the case. Other counties are also represented. The brunt of the work, however, is being carried by E. J. B. Schubring, of Madison, Wis., who has specialized in engineering law and practice, is familiar with engineering technical terms and is admirably equipped for the conduct of a case like this.

The principal attorneys of the opposition are Percy Taylor, of Sidney, representing Miami and Shelby counties; J. A. Kerr, representing Tippecanoe City; F. C. Goodrich, prosecuting attorney of Miami County; and C. L. Darlington, representing Greene County.

Nothing so mystifies an onlooker at the hearing, even one who has kept in close touch with the history of the case, as to try to discover the real basis of opposition to the conservancy plans. That opposition has been strenuous and resourceful from the beginning, but just what it is based on is difficult to say. In Mr. Taylor's opening address, following Mr. Schubring, he professed to disclose the grounds of the opposition, but except for a single reference to the public health of the community the sole ground presented was the lack of safety in the dams. There was no dissent from the proposition that something should be done. "We all want to get together," said Mr. Taylor, "and see what is the best thing to be done for the district." Even cost was not the matter "of prime importance," but safety, and Mr. Taylor continued:

The fundamental proposition is this, as we see it. Is the plan of protection by retarding basins safe and feasible, or is it not? Now, that is absolutely the crux of the whole situation. Anything else is practically surplusage. Of course, there are questions of local flood protection that ought to be heard and will be heard, but the thing we want to get at is this one matter; everything hangs upon that: Are the dams safe?

Elaborating this idea, the whole system of retarding basins was attacked in general and in detail as essentially unsound and creating an ever present menace to the inhabitants in the valleys below.

Unfortunately this line of attack lacks the essential elements of sincerity. Those who are mainly using it live *above* the dams, except some few below the small basin at Lockington. No complaint worth considering has come from *below* the main dams. There is no pronounced distrust of their safety there, where if anywhere it must exist. It is therefore impossible to accept this argument as the real reason. It is effective, it strikes a popular chord, it plays upon public fears and is a useful instrument in accomplishing the real purpose. Yet what the real purpose of the opposition is and what is the motive back of it are nowhere definitely disclosed.

The writer is convinced in his own mind that it is the desire to escape the burdens of an assessment district, which some fear will be out of proportion to the benefits. Indirectly this idea has cropped out in numerous places. Then there is, or has been, a feeling that the scheme is for the benefit of Dayton, Hamilton and other towns and that the outlying districts will be taxed for their benefit. Finally, the very fact of opposition has gained a certain vogue, enters more or less into local politics, serves to sustain much legal practice, and thus its continuance derives an importance in certain directions which is not to be lightly estimated.

Such is a general view of the situation as nearly as the writer has analyzed it. He has not been able to lay his finger on what he can term a single valid objection. There may be such; but if so they have been covered up under the plausible appeal of safety, which, as already stated, lacks the simplest element of sincerity.

Maneuvering for Advantage

Inasmuch as the hearing was for the sole purpose of considering objections to the plan, it was assumed as a matter of course that the objectors would open the case by the introduction of their own witnesses. Not so. The opposition asked to be allowed to call Chief Engineer Morgan of the Conservancy District as an adversary witness for purposes of cross-examination. After some deliberation the court granted this request, and Mr. Morgan took the stand.

For five consecutive sessions (five days) Mr. Morgan was on the stand undergoing a continuous fusillade of questions. His testimony brought out in the clearest possible light the magnificent work that he and his able staff have been doing during the past three years. There was no point so obscure, no question so abstruse, no item of data so unexpected that the required information was not instantly at hand. Friend and foe alike recognized his splendid achievement, and his testimony, taken altogether, was an education to most of those present. Day after day the opposition beat itself against this wall of information only to recoil without making the slightest impression.

Some of these encounters had an element of humor about them which even the dignity of the courtroom could not suppress. One lawyer, a gentleman of much solemnity, thought he had at last got the witness where he wanted him. He had heard the witness state that the average velocity of flow of the river at the crest of the 1913 flood was about 4 mi. per hr. (including the overflow section), and it seemed to counsel so incredible that he set about to refute it. At last he found what he wanted. Brandishing a report of the United States Weather Bureau, he declared that this authority proved that the velocity between Dayton and Hamilton was 17½ mi. per hr. Following is an excerpt from the colloquy that ensued:

Q. How did it happen that it [the crest of the flood] was two hours after it was here? Was it because of the fact that it took it two hours to run from here to Hamilton? (Laughter)

A. Not at all.

Q. And didn't the stream pass down at the rate of 17½ mi. per hr.? (Laughter.)

A. If it had, there would be a hole in this valley many feet deep from here to Hamilton.

Q. I am not asking anything about results. Isn't it a fact that that is what it did?

A. It didn't do anything like that.

Q. It didn't do anything like that?

A. Not at all.

Q. Then the Weather Bureau report is not correct?

A. The Weather Bureau made no such statement.

Q. What did they state?

A. I am speaking from memory. The Weather Bureau stated that the crest at Hamilton occurred only about two hours after the crest at Dayton.

Q. Doesn't that mean that it took it two hours to run here to Hamilton? (Laughter.)

Mr. Morgan's testimony fills 620 typewritten pages. No adequate summary is here possible, and only a brief notice of some of the leading features can be presented.

A very complete outline was given of the plan itself and a clear description of building dams by the hydraulic process. The manner of handling floods during construction and of effecting final closure of the dams was gone into at length. Danger of settlement in hydraulic-fill dams was shown to be very slight. The causes of the accident to the Necaxa dam in Mexico were explained, this accident being the chief reliance of the opposition in its effort to discredit the hydraulic-fill process. Many earth-dam failures were cited, their causes analyzed and shown, in practically every case, to be avoidable.

Mr. Morgan explained in this connection how great a factor of safety (even bordering on the extreme) had been adopted for the proposed Miami dams. Wave-wash, muskrat action and other sources of trouble were shown to be nonexistent in basins that are empty practically the year around.

The danger of clogging the conduits was pressed by the opposition more than any other one point. Mr. Morgan clearly showed that it amounted to practically nothing.

He described in detail the form of drift barriers provided in the official plan, showed that ice could never come in volume or size of blocks that would choke the conduits; that the basins themselves were to be kept free of floatable material; that drift of any considerable size brought down from above could only come after a flood had reached a point that would fill the basin well over the conduits, converting them into lakes without current velocity and on which drift would be driven about by the wind and more likely than not be stranded on shores; and that in any case a little intelligent care at the proper time would take care of any drift that might come against the barriers.

But no argument would avail with the opposition. Visions of great sycamores, highway bridges and houses hurled down by a raging torrent into the vortex of these conduits, forming there a close grillage that would quickly be sealed up by haystacks, corn fodder and trash of all sorts, were held up to the court as something absolutely certain to happen if the dams were ever built.

Mr. Morgan went very thoroughly into the matter of channel improvement and, for that matter, into every other feature of the subject. But space permits no further consideration of his testimony here. If the opposition thought to secure any tactical advantage by putting Mr. Morgan on the stand in the first instance, it is difficult to see where it succeeded.

Mr. Hill's Testimony

Following Mr. Morgan, John W. Hill, of Cincinnati, was called by the opposition for direct examination. Mr. Hill had a contract with the three counties of Miami, Shelby and Clark to present their case at the hearing. He was the only engineer placed on the stand on that side of the case. He had prepared a report of over 150 printed pages, which was introduced in evidence, and his own testimony was in elucidation of this report.

Concerning both report and testimony, the most charitable commentary that can be made is that they were unfortunate, whether from a professional point of view or as a contribution to the cause of the opposition. It is true that Mr. Hill's time and resources, as he repeatedly urged, were very limited, and it was not to be expected that he could offset in wealth of detail the elaborate work of the district itself. But the report was not merely weak in this respect; it disclosed a willingness to adjust data to suit the exigencies of the case, and this feature was so manifest as to deprive the report of all claim to public confidence. A few examples will show this general tendency.

Mr. Hill sought to attack the hydraulic-fill method of dam construction on the authority of the late J. D. Schuyler, its most earnest champion, if not the father of the method itself. In doing this he quoted Schuyler's well-known book, but only as already quoted in a trade publication on earth dams put out by the Ambursen Co. As a matter of fact, he quoted Ambursen a good deal more than he did Schuyler, but in a way that left the impression that it was all from Schuyler. At the same time he omitted many citations that he might readily have taken from Schuyler, showing that gentleman's unqualified faith in the hydraulic-fill method.

Bearing in mind the fact that the Ambursen Co. is exploiting on the market a type of hydraulic-fill dam in opposition to the ordinary type (but as yet wholly untried in practice), the utter unfairness of these citations from Ambursen, quite aside from the indefensible course of crediting them to Schuyler, is apparent.

In the very important matter of cost estimates the same policy of unfairness appeared. Unit prices of 12c. and 14c. for excavation were used in the Hill report for working out the cost of his proposed channel improvement, while in revising the estimates of cost in the official plan, where 30c. per cu.yd. is used for the same class of work in the same localities, Mr. Hill increased this 30c. to 75c. and then made comparisons between the two estimates as if they were made up on the same basis. If the same unit price is applied to the Hill estimate that he uses in revising the official-plan estimates, the cost of his channel-improvement scheme will be increased from about $40,000,000 to about $200,000,000.

In estimating the necessary channel improvement through Dayton under his own plan Mr. Hill assumes a runoff for Mar. 25, 1913 (the date of the flood crest), of 105 sec.-ft. per sq.mi. of watershed, a material reduction from the recorded discharge. In another place, estimating probable flood conditions at Hamilton with reservoir control, he adopts for the watershed below the reservoirs a figure of 125.6 sec.-ft. per sq.mi., or the full depth of rainfall as given by him for that date. He assumes the dams to be filled to capacity (which would require a storm more than twice as great as that of 1913); makes no allowance for retardation of basin discharge, but makes it arrive simultaneously with the uncontrolled runoff, and thus deduces a figure of 356,000 sec.-ft. discharge, almost as great as his own estimates below Hamilton (Hill's report, page 12) without any reservoir control whatever. As a matter of fact the controlled flow at Hamilton for a 1913 flood will be between 175,000 and 200,000 sec.-ft.

Passing by these inconsistencies, which abound throughout the report, a feature of Mr. Hill's scientific methods may be noted. He was unable to make a survey of the entire river to determine the cost of channel improvement. He had, however, made such a survey and estimate through Butler County immediately after the great flood. To apply this work to the whole river, Mr. Hill assumes that "nature has demonstrated" that the size of channels is proportionate to the ordinary flood flow; that this flow at any point is proportionate to the area of watershed above it; and that the cost of enlargement is therefore directly proportional to such area and to the length of the particular stretch of channel considered. Thus the excavation in Butler County as found by his survey was 87,000,000 cu.yd.; the channel length, 27 mi.; and the average watershed above, as 3,222 sq.mi. Dividing the yardage by the product of 27 and 3,222, he obtains a coefficient of 1,000, which he applies successively to seven other stretches of the river and thus deduces the channel excavation for the whole stream necessary to carry a 1913 flood.

Mr. Hill's main contention was that channel improvement would answer every purpose and that it was cheaper than the official plan. On this last point he failed completely, and his whole argument was finally whittled down to the single point that such great dams ought not to be built so near large centers of population. Under cross-examination he was unwilling to commit himself absolutely to the proposition that a hydraulic-fill dam could not be made safe. If earth dams were to be built, however, he preferred the old method of spreading and rolling. But he thought that no dams should be built where these are proposed. This was the final result of his report and

testimony—a personal dictum rather than a scientific conclusion.

A feature of the opposition's case which obtruded itself on all occasions was the assumption of conditions that could never occur. There is not the remotest possibility of any storm ever filling these dams to the spillways, to say nothing of the crests of the dams. Yet this last impossible contingency was constantly assumed and, by continuous reiteration, no doubt created a feeling that it might possibly occur.

Over and over the opposition insisted upon assumptions that virtually amounted to this: What will happen if a storm occurs of such magnitude as to exceed the power of the dams to resist it? The answer must be that in such a case the dams would fail. This sort of argument no doubt had a certain effect upon some. Its chief virtue was that it shut off all counter-argument.

Additional Testimony

In the minds of most of the attendants at the hearing, the case was practically settled by the testimony of Morgan and of Hill. The opposition, however, called several minor witnesses on purely local features of the plan, and then the district took up its own direct examination.

The first witness called was E. A. Deeds, president of the Board of Directors. In straightforward and simple language Mr. Deeds told the story of the work for flood protection in the valley, in which, from the beginning, he has borne a leading part. His statement of the public purpose back of the whole movement and of the manifold difficulties that had beset the work on every hand made a profound impression upon court and audience alike. There was no cross-examination.

Following Mr. Deeds, Chief Engineer Morgan was called. Although he had gone over practically the whole ground on his cross-examination, it had been done in such a desultory and disconnected fashion that it was considered best to place in the record a consecutive and orderly narrative of the work of his department. The opportunity was improved to bring out certain points passed over on cross-examination. Mr. Morgan is still on the stand at this writing (Oct. 26).

Following Mr. Morgan, several members of his staff will be called on special features of the plan. Then will come the witnesses brought in from the outside. These include T. W. Jaycox, of Denver, Colo., and F. J. Fischer, of Los Angeles, on the hydraulic-fill method; G. B. Massey, of Chicago, on methods and costs of channel improvement; and J. W. Alvord, of Chicago, Prof. D. W. Mead, of Madison, Gen. W. H. Bixby, of Washington, and the writer, upon the general features of the plan as a whole. The object in presenting this rather formidable array of testimony was apparently not so much because it was considered necessary for the purpose of this hearing, as it was to make up as complete a record as possible in the event of an appeal from the findings of this court.

Comment

Viewing the case as a whole, it well illustrates what the writer has often remarked: The greatest obstacles that the promoters of public work have to overcome are not those of nature, but of man. Nature is sometimes a stubborn adversary, but she always acts in the open, without subterfuge or indirection. But human ignorance, prejudice and self-interest are handicaps of a different character. Ignorance is least important, because it may yield to instruction. Prejudice—that is, prejudgment of a case and then sticking to it regardless of facts—is immeasurably worse. But self-interest is the most insuperable obstacle of all. Public measures are judged by their effect on the private pocket-book, and the rarest phenomenon in the world is a willingness to subordinate personal interest to the public welfare.

It is obstacles like the foregoing that the Board of Directors of the district have been and are still being called upon to overcome. The engineering problem is well worked out. No such thorough investigation was ever before made of an engineering problem. The cost of the work to date is something over three-quarters of a million dollars. Even the Panama Canal is not an exception.

The engineering staff is ready to proceed with construction. But months and perhaps years must yet be wasted in this exasperating struggle with purely artificial obstacles. It is important that the people of the valley, who really want protection, understand this situation as it is. Any further delay in proceeding with the execution of the work will not come from the merits or demerits of the plan itself, or any lack of preparation to proceed, but solely from an intangible opposition such as is described above. From the point of view of the physical problem, the work ought to be entirely completed within the next five years.

Water Meters Give Remarkable Results at Redondo

By Edward R. Bowen*

During the summer of 1916, meters were installed on all services of the Redondo Water Co. at Redondo, Calif. This company serves about one thousand consumers. The decrease in the consumption of water resulting solely from the installation of meters is striking, as is shown by the accompanying tabulations from the superintendent's monthly reports.

REMARKABLE EFFECT OF WATER METERS, REDONDO, CALIF.

	June		July		August	
	1916 Metered Service	1915 Unmetered Service	1916 Metered Service	1915 Unmetered Service	1916 Metered Service	1915 Unmetered Service
Time pumps operated, hr.	304½	923	325¼	985½	336	944½
Amount of power consumed, kw.-hr.	11,430	33,240	12 560	34,930	12,930	35,600
Amount of water raised, gal.	13,183,700	40,105,400	14,201,000	42,822,000	14,654,200	41,039,500
Amount of power consumed per 1,000 gal., kw.-hr.	0.8670	0.8288	0.8844	0.8157	0.8823	0.8674
Cost of pumping per 1,000 gal., dollars	0.0171	0.01336	0.01687	0.01328	0.01665	0.01377
Amount per capita per day pumped, gal.	109.86	334.21	114.52	345.34	118.18	330.96
Rainfall during month, in.	0	0	0	0	0	....
Amount received from consumers per 1,000 gal., dollars	0.1598	0.0472	0.162	0.0484	0.1725	0.0505

*Engineering Offices of J. B. Lippincott, Central Building, Los Angeles, Calif.

Criticisms of Toronto-Hamilton Concrete Road Answered

The recently constructed concrete pavement of the Toronto-Hamilton highway in the Province of Ontario, Canada, has just been made the subject of unfavorable criticism by R. C. Harris, Commissioner of Works of the province. A considerable number of longitudinal cracks and raised joints were reported in August, 1916, less than a year after most of the pavement had been laid. The condition of the pavement was of peculiar interest because apparently every effort had been made to insure a first-class job, and the question naturally arose whether concrete was a suitable paving material for the extreme climatic conditions of eastern Canada.

The chief engineer of the Toronto and Hamilton Highway Commission, H. S. Van Scoyoc, was asked to make a personal inspection of every slab and joint in the 162,841 sq.yd. of concrete pavement laid in 1915, and his report to the commission, an abstract of which is given herewith, contains some interesting information on defects of this pavement and their probable causes. A general summary of the results of Mr. Van Scoyoc's inspection follows:

GENERAL SUMMARY OF CONDITION OF PAVEMENT

SLABS

Total number		2,076
	No.	%
Uncracked	1,751	84.3
Slightly cracked	243	11.8
Total	1,994	96.1
Cracks need filling	82	3.9

JOINTS

Total number		2,071
	No.	%
Satisfactory	1,994	93.9
Need chipping	85	4.1
Need chiseling	42	2.0

SUMMARY; WORK DONE SUBSEQUENTLY TO OCT. 15

SLABS

Total number			654
% of total number laid in 1915			31.5
	No.	%	% of Total Laid in 1915
Uncracked	526	94.9	30.0
Slightly cracked	95		39.1
Need filling	33	5.1	40.2

JOINTS

Total number			654
% of total number laid in 1915			31.5
	No.	%	% of Total Number Laid in 1915
Satisfactory	548	83.9	28.2
Need chipping	65	10.1	76.4
Need chiseling	41	6.0	97.6

SURFACE DEFECTS

Total number of square yards	162,841 = 1,465,569 sq.ft.
No. of holes noticed	1
Size	4x4 in. = 16 sq.in. = 0.11 sq.ft.

Chief Engineer's Conclusions

The summary shows quite conclusively that the trouble developed in those sections laid after Oct. 15. It is open to serious question if any work should be done in this locality after that date. The temptation was great last season, both on account of the extremely wet weather earlier, which had delayed progress, and the unusually mild weather, which continued well into November. Careful construction succeeded in preventing any injury to the surface by the frost, but it appears that the work was not so fortunate so far as joints were concerned.

The explanation of the trouble is believed to be that concrete expands not only with an increase of temperature, but also with an increase in the quantity of moisture it contains. It contracts not only with a decrease in temperature, but also as the excess water used in mixing dries out. Ordinarily, concrete is laid in warm weather. It contracts in setting up and in addition contracts with the cooler weather during the winter months. The slabs shorten more than enough to provide for the increase in length with the rise in temperature during the following summer.

In this case the concrete laid late did not dry out, on account of the low temperature. A very wet fall was succeeded by an extremely wet, cold spring. The warm weather came suddenly, and the concrete expanded before there had been the usual contraction. Even with these conditions it is believed that there would have been no noticeable trouble if the joints had been vertical.

All the joints that have been chiseled through are more or less inclined from the vertical. In practically every case this inclination is in the direction in which the mixer was working. The only exceptions that have been found up to the present have been on the steeper grades, where the tendency of the concrete is to flow down hill, and in the joints made at the end of a day's work, where there was probable lack of care.

It is quite possible that wider joints would have lessened the trouble, but in this connection it should be added that to have increased the thickness of joints to ½ in. would have entailed an additional expenditure of more than $5,000 for materials alone. It is to be anticipated that the maintenance of a wide joint would be greater than that of a narrower one. In this case a small part of the $5,000 will put the joints in such shape as to cause no serious inconvenience to travel.

The longitudinal and other cracks have been tarred on the approximately 7 mi. from Oakville Bridge to the Blue Dragon at a cost of less than $25 for materials and $175 for labor. This includes any tarring that has been done on the joints as well. Four or five more barrels of tar were required for the remaining work.

Mr. Van Scoyoc further states, in a communication to the editor, that less than 4% of the total number of slabs required attention from cracking—a condition which compares favorably with concrete highways of any extent that have come under his personal observation or knowledge. He says that there was reason for criticism regarding a limited number of joints, more particularly during the early summer; but this condition, however, to a large extent, righted itself, and a very limited amount of repairs has removed grounds for this complaint.

The New Pier at Cristobal, Canal Zone, although not quite completed, has been put to use for unloading cargoes to relieve the pressure for more handling space at the other two piers, Nos. 8 and 9, and Dock 10. The new pier, No. 7, is 1,035x212 ft. Of the 237,300 sq.ft. of floor space, 155,000 is inclosed and roofed. The pier is carried on 212 steel cylinders 6 ft. in diameter and 125 ft. long, with tops at water level. They were sunk by open dredging and filled with concrete in which are embedded steel rails for carrying girders for the floor system, composed of continuous steel stringers with 11-in. concrete slabs. The shed is of steel framework with concrete walls and cement-tile roof, the 21,000 pieces of tile being furnished by the American Cement Tile Manufacturing Co. Pier 7 has no depressed railroad tracks running the whole length through the middle, as is the case in Piers 8 and 9; but it has a railroad track running outside along the walls, for handling freight. In the construction of the shed 2,200 tons of steel was used. The total cost of the pier is $1,420,000. With its completion there will be available at Cristobal 581,150 sq.ft. of floor area in the new concrete dock and pier system at the Atlantic terminal of the canal, of which area 418,180 sq.ft. will be inclosed. To provide for the still larger demand expected within the next few years, preliminary work has already been begun and the contract let for the steel cylinders for another concrete pier of the same size as No. 7, to cost about $1,500,000 and to be completed within two years

News of the Engineering World

Transit Head for Pittsburgh

Major Armstrong, of Pittsburgh, Penn., on Nov. 6 sent to the City Council for confirmation as Rapid Transit Commissioner the name of Edwin K. Morse. The position, which pays $8,000 per year, was recently created by an ordinance which directs the commissioner to make a transit survey of the city and to recommend a solution of the transit problem. A budget is ordered as soon as possible. Mr. Morse is a Pittsburgh engineer who has devoted most of his attention to bridge work. He was born in 1856 and graduated from Yale in 1881. His early professional experience was in bridge shops and in 1887 he went to Australia to erect the superstructure of the famous Hawkesbury Bridge there. Since his return in 1889 he has resided at Pittsburgh and has had charge of the construction of a number of the bridges in that district.

Bethlehem Steel Co. To Spend $60,000,000 on Extensions

The Bethlehem Steel Co., in a circular signed by its President, E. G. Grace, under date of Oct. 30, announces that it will expend approximately $60,000,000 in the next four years in extending the steel plant which it purchased from the Pennsylvania Steel Co., and in diversifying products. The circular says:

> Additional blast furnaces will be built at both Steelton and Sparrows Point. The finishing mills at Steelton will be enlarged and extended and new lines of products added. At Sparrows Point advantage will be taken of its exceptional position as a point for manufacturing material for export; and besides the improvements and extensions to the existing rail mills, there are in course of erection mills for the manufacture of merchant bars, rods and sheets, a general commercial line of plates and a complete tin-plate mill.

Easton-Allentown Concrete Road Open for Traffic

The 8-mi. concrete road completed this season between Easton and Bethlehem, Penn., was formally opened with appropriate ceremonies on Thursday, Nov. 2. The celebration was in charge of the William Penn Highway Association of Pennsylvania and the Portland Cement Manufacturers Association, with the coöperation of some of the civic and social organizations of Easton and Bethlehem. Special trains were run from New York, Philadelphia and Harrisburg, bringing over 1,000 guests. An automobile parade with nearly 1,500 vehicles in line went over the new road from Easton to Bethlehem. Here a barrier across the road was removed by the guest of honor, Governor Brumbaugh of Pennsylvania, assisted by two young ladies, while a third broke a bottle of champagne upon the road surface. Luncheon was served at Oak Park, a resort midway between Bethlehem and Easton. Addresses were made by Governor Brumbaugh, Congressman Henry J. Steele and W. D. Uhler, Chief Engineer of the Pennsylvania Highway Department. Governor Brumbaugh announced that he would urge the passage by the next legislature of a state appropriation of $5,000,000 to build concrete roads in Pennsylvania, and Chief Engineer Uhler stated that the policy of the State Highway Department hereafter would be to spend the state money only for roads in which concrete was used either for the entire road or for the foundation. Methods of work in the construction of the road have been described in past issues of *Engineering News*. The finished road received high commendation from the many highway engineers who examined it, about the only criticism made being that its width, 16 ft., is too narrow for fast automobile traffic. It is probable that the remaining section to be built next season from Bethlehem to Allentown will be 18 ft. wide.

Los Angeles May Be Its Own Contractor--If

By an amendment to the city charter of Los Angeles, Calif., approved by popular vote on Oct. 24, the city is authorized to make various street improvements by the day-labor or by the contract plan, as it sees fit, and also to assess the cost of the improvement upon either abutting property or upon an improvement district. The specific work that may be so done and assessed is "the construction or reconstruction in any public street, alley or other public place, or in any right-of-way owned by the city, of sewers, drains, water or gas mains, and line and conduits for transmitting electric energy, and other pipes, mains, lines and conduits, or other public improvements."

Ordinances providing for such improvements are made liable to suspension for a period of not less than six months on protest by owners of half the frontage affected, unless meanwhile a majority of the frontage petitions for the improvement; but such protests will not be effective as regards sanitary sewers or certain minor improvements adjacent to similar existing adjacent improvements.

A restrictive "if" on the day-labor plan is included in the charter amendment, as follows:

> No improvement to be paid for by special assessment shall be made by the city by the direct employment of labor and purchase of materials, under the authorization of this subdivision, unless the city shall have been requested so to do by a petition signed by the owners of a majority of the frontage to be assessed for such improvement, or unless at the time of receiving bids for the construction of such improvement the city shall have submitted a sealed bid and been the lowest responsible bidder; provided, that should any public improvement provided for in this subdivision be advertised to be let by contract and the bids for the same be, in the opinion of the owners of a majority of the frontage to be assessed for such improvement excessive, then upon application to the council by the owners of such majority of frontage, duly made within 20 days after such bids have been received, all bids may be rejected and the city may proceed to construct the proposed improvement by the direct employment of labor and purchase of materials.

The vote (unofficial) on the amendment was 24,517 for to 22,450 against.

ENGINEERING SCHOOLS

Harvard-Technology

A fellowship for research study of asphaltic materials and their uses, to be known as the "Clifford Richardson Fellowship," has been established in these associated institutions. The appointment of the incumbent of the fellowship and the choice of subjects for investigation, as well as the disposition to be made of the results of such investigations as may be undertaken, are to be decided by the institute faculty or the joint committees of the university and the institute having control of engineering work. The funds were provided by the Barber Asphalt Paving Co., of Philadelphia.

University of California

As part of its course in mining engineering under Prof. Frank H. Probert, the University of California has started construction of the "Lawson Adit," an 1,800-ft. tunnel into the Berkeley Hills, starting just east of the Hearst Memorial Mining Building. The object of this work is to familiarize the students with all the mechanical processes of mining, and classes will be held in the underground workings.

University of Tennessee

The University of Tennessee has organized its extension courses for the benefit of shop employees and others who may be benefited by practical courses in mechanic studies. This is the second year of this work. The classes are held at Memphis, the largest city in the state, under the direction of Prof. Charles E. Ferris, dean of engineering. The total cost is $5 and in some instances 60-day notes are accepted in payment. Thomas H. Allen, consulting engineer, is assisting. The Memphis Engineers' Club is coöperating. The State Manufacturers' Association last year paid part of the cost. Correspondence courses are also given.

Massachusetts Institute of Technology

The first of the dormitories at the site of the new Technology buildings in Cambridge have been opened. These are cream-colored buildings in the Florentine style, located a little below the educational group facing the Charles River Basin. The group is L-shaped in plan with a tower in the angle, and forming a block of six independent houses, each with its own entrance, stairway and party walls. The latter are pierced by fire doors to be unlocked only in emergencies and after breaking a glass guard. The two end houses have been assigned to two fraternities which have desired to experiment with quarters in the blocks about the campus. These houses have different layout from the others—each having a kitchen, dining room and two large living rooms. The restaurant and Students Union, serving the regular dormitories, are temporarily located in the basement—pending construction of the Walker Memorial Building. Student government will rule these dormitories, there being a faculty committee having general supervision of such matters as sanitary inspection, maintenance, etc.

University of Illinois

The College of Engineering of the University of Illinois is to have the benefit this year of an Advisory Board for the Railway Industries of Illinois, which upon the recommendation of the dean and the approval of the president of the university, has recently been appointed by the trustees. The presidents of five of the larger railroads centering in Chicago, and having their administrative offices in that city, have consented to serve as the members of this Advisory Board. The Department of Railway Engineering is soon to receive a modern high-power superheating locomotive for use during the fall and winter under the direction of Professor Schmidt.

Initial steps have been taken and authority has been received from the president and the trustees of the university, which provide a foundation for the establishment within the College of Engineering, in coöperation with the State Public Utilities Commission of Illinois, of an Electrical Standards Laboratory to which the public service corporations of the state can send any piece of apparatus to be checked and corrected.

NOTES

A Gravel-Bank Cave-In killed five men at Barton, Ala., Oct. 24. The men were taking out material for roadwork and were buried by the fall of several tons of gravel.

A Mine Explosion at Marvel, Ala., on Oct. 22 caused the death of every man in the mine of the Roden Coal Co., except one—17 men. The one who escaped was a pumpman in one of the upper headings. All the men killed had gone into the mine to install a new electric-feeder cable. Open lights were used, but up to this time the mine had not been considered gassy. The United States Bureau of Mines mine-rescue car was sent to the scene of the explosion, but by the time it arrived gas helmets were no longer necessary.

Three Adjoining Brick Buildings Collapsed on Portland St., Boston, Mass., Oct. 26. The original failure appears to have been in a six-story brick structure at 21-37 Portland St. In falling, it brought down the two adjoining buildings, a one-story and a three-story structure. The six-story building was unoccupied and was being remodeled. The collapse occurred about 8 p.m. During the day about 50 workmen had been employed on the remodeling. No lives were lost, but the loss in property is stated to amount to many thousand dollars.

A Boston Surface Car Jumped an Open Drawbridge killing 50, on the evening of Nov. 7. The car, jammed with passengers from South Boston, Mass., smashed through the gates in front of the open draw in Summer St. and plunged into Fort Point Channel. Press dispatches state that the accident appears to have been caused by the absence of a red lantern on the guard-gate and the fact that the electric arc lamp that usually illuminates the bridge was not lighted. A floating crane was installed at the open span, and divers went down to the wrecked car, which they reported was standing on end on the bottom. The motorman and conductor escaped death by jumping. They have since been arrested on technical charges of homicide.

The Breakage of a Locomotive Driving Axle, which caused a train accident on the Southern Ry. June 7, 1915, has been investigated by the Division of Safety of the Interstate Commerce Commission. Examination of the broken axle was made by James E. Howard, Engineer-Physicist, and it was ascertained that the cause of the fracture was an area of metal at the surface of the journal which contained chatter marks and short incipient cracks made by the roughing cut in the lathe during the machining of the forging from which the axle was made. It appeared that the finishing cut was not deep enough to take off all the metal which had been injured by the roughing cut. Defective workmanship on the axle is further indicated by the fact that the length of bearing of the axle in the wheel-seat was from ½ to 1¾ in. long, instead of bearing for the whole length of 8 in. in the wheel-seat as it should have done. Two other fractures had started in the wheel-seat which would soon have caused failure of the axle had it not broken in the journal.

Paving By Day Labor in the City of Minneapolis, Minn., amounted to 400,000 sq.yd. in the season just closing. This is approximately 25 mi. of streets. Three-quarters of the total was creosoted wood-block pavement for which the city has become famous. E. R. Dutton is assistant city engineer in charge of paving.

Railway-Electrification Work on a large scale is planned by the Great Northern Ry., according to reports from the State of Washington. The company has planned to alter to electric traction all its lines in the State of Washington, at an estimated ultimate cost of $16,000,000 to $20,000,000. It is stated that the water-power sites where current will be generated to operate the lines have already been chosen.

The Panama Canal during the first two years of its operation, from Aug. 15, 1914, to Aug. 14, 1916, was used by 2,097 ships, with an aggregate net tonnage (Panama Canal measurements) of 7,046,407. These vessels carried 9,031,613 gross tons of cargo. During the second year of operation the canal was closed for nearly six months, which reduced the amount of traffic through it nearly one-half. The total tolls collected were $5,102,000 in the first year and $2,310,000 in the second year.

A Municipal Ocean Steamship Line has been recommended to the mayor and council of Los Angeles, Calif., by the Chamber of Commerce. It is stated that Los Angeles exporters have gone to great expense in extending the city's trade, but find themselves handicapped by conditions over which they have no control. The city charter, adopted by popular vote, allows the city to engage in the shipping business, and the Chamber of Commerce suggests that a start be made by building two wooden ships at a cost of not over $150,000.

A National Conference to discuss uniform state legislation covering the construction of boilers has been called by the Industrial Commission of Ohio, to take place at Washington, D. C., during the week beginning December 4. It is probable that the Boiler Code of the American Society of Mechanical Engineers will be the basis for discussion at this meeting. It is stated that the City of Philadelphia is about to adopt a new boiler ordinance based on the A. S. M. E. Code to replace its old rules regarding boiler construction, which are said to have been in existence since 1868.

The Union Railway Station buildings in St. Paul, Minn., were condemned by the State Fire Marshal, R. W. Hargadine, on Oct. 20 and ordered to be removed within six months. The order was issued on the ground that the buildings were dangerous to human life and surrounding property. Plans for the construction of a new station by the St. Paul Union Depot Co., of which W. C. Armstrong is Chief Engineer, have not yet been agreed to by all the roads concerned. The plans for a new station first prepared have been objected to by S. M. Felton, President of the Chicago Great Western R.R., on the ground that the plan of operation of the track system would be prohibitively expensive.

Municipal Ownership of the electric light and power distributing systems of Los Angeles is likely to be realized in the near future. Negotiations have been in progress for months between the city authorities and the three companies distributing electric current within the city limits. The city has offered the companies $10,000,000 for their distributing systems and the steam plant of the Los Angeles Gas and Electric Corporation. The companies have offered to sell the property at $13,691,000. The city also offers to purchase power from the companies for 5½ yr. at 0.85c. per kw.-hr. The companies ask for a 10-yr. power contract from the city, during which time the city would purchase power from them for half its requirements at $33.17 per hp. The city is already at work paralleling the companies' distribution systems, but this work will of course be stopped if an agreement is effected for the sale to the city of the existing lines.

Reduced Rates for electric current have been offered to the Public Service Commission for the First District of the State of New York by the New York Edison and United Electric Light and Power Co., and would apply to the Borough of Manhattan and part of Bronx, beginning Jan. 1, 1917. The maximum tariff in both the general lighting-rate and power schedules is reduced from 8 to 7½c. per kw.-hr. until July 1, 1917, after which time it is further cut to 7c. The right is reserved to restore the 8c. maximum if unforeseen conditions adversely affect the companies' financial status. This is reported to mean a revenue reduction of $1,750,000. Similar reductions had previously been ordered upon the associated Brooklyn Edison Co., and this offer was the outcome of conferences between the Public Service Commission, the city administration and the companies. The change is acceptable to the city government and it is expected that it will be formally accepted by the Commission.

The Hudson Bay Railway enterprise has been under investigation by the Canadian Government Railway Commission, which heard at Ottawa the testimony of W. A. Bowden, Chief Engineer of Construction in the Department of Railways and Canals. The theoretical advantage of the Hudson Bay route is the saving in transportation cost on grain exported to Europe. The distance from the principal grain-raising centers in the Canadian Northwest to Port Nelson on Hudson Bay is practically the same as the distance from Winnipeg to Fort William at the head of Lake Superior. The sailing distance from Port Nelson to Liverpool is substantially the same as from Montreal to Liverpool, and it was computed that the saving in the transportation cost by reason of the shorter land journey would be from 5c. to 12c. per bushel of wheat. Mr. Bowden further said: "The information we had when we started the work has proved very unreliable during the five seasons we have been working. For example, our information was that Hudson Bay would be open to navigation about the first of July. The fact is it is not open until the last of July or the first of August. The date when navigation was closed by ice was not fixed in that early information. However, ordinary tramp steamers have left Port Nelson on Oct. 23 and gone through without damage. A Government tug went through on Nov. 1, but the ice would have damaged a tramp. The grain shipped in August by this route would have to be what was left over from the previous year's crop. New grain could be shipped in September and October.

PERSONALS

L. D. Reilly has been appointed Manager of the Ambridge plant of the American Bridge Co., succeeding F. T. Cadmus, who has resigned.

Elmer Folsom and **Milo C. Taylor** announce the formation of the firm of Folsom & Taylor, Civil Engineers, Peoples Bank Bldg., Bloomington, Ill.

Francisco Asenjo, Civil Engineer, of Santiago, Chile, is making an investigation of irrigation systems in the West. He is now in California.

Sanford E. Thompson, M. Am. Soc. C. E., Consulting Engineer, has established offices and a concrete-testing laboratory at 136 Federal St., Boston, Mass.

G. C. Habermeyer, of Aurora, Ill., has been appointed Acting Engineer of the Illinois State Water Survey, with office at the University of Illinois, Urbana, Ill.

Ealy G. Briggs, Assoc. M. Am. Soc. C. E., formerly Inspector, Department of Public Works, St. Paul, Minn., has been appointed Sidewalk Engineer of the department.

Charles A. Lemmon, formerly Chief Engineer of the Butte, Anaconda & Pacific Ry., is now Assistant to the General Manager of the Anaconda Copper Mining Co., Anaconda, Mont.

A. Leckie, former Division Engineer of the Arkansas Western Ry., at Pittsburgh, Kan., is now Division Engineer of the Kansas City Southern Ry., at Kansas City, Mo., succeeding E. M. Basye, transferred.

M. F. Clements, M. Am. Soc. C. E., Assistant Engineer of the Northern Pacific Ry. at Spokane, Wash., has been promoted to be Bridge Engineer, succeeding H. E. Stevens, who was made Chief Engineer, as noted in these columns Sept. 28.

Charles T. O'Neal, formerly Superintendent of the Lehigh Valley R.R. at Buffalo, N. Y., has been promoted to be General Superintendent, with headquarters at South Bethlehem, Penn. He is succeeded as Superintendent at Buffalo by P. J. Flynn.

Wendell P. Ball, recently with the valuation department of the Cincinnati, Hamilton & Dayton Ry., at Cincinnati, Ohio, has been promoted to be Assistant Engineer in the maintenance-of-way department, with headquarters at Cincinnati.

H. H. Barber, formerly Sales Engineer, and W. B. Greene, formerly Advertising Manager of the Stephens-Adamson Manufacturing Co., Aurora, Ill., announce the formation of the Barber-Greene Co., manufacturers of car unloaders, wagon loaders, etc., Aurora, Ill.

Edward Ray Weidlein has been appointed Associate Director of the Mellon Institute of Industrial Research of the University of Pittsburgh. He was graduated from the University of Kansas in 1909 and has been a senior fellow in charge of hydrometallurgical investigations at Mellon Institute.

W. H. Vance, recently Engineer of Maintenance-of-Way of the Louisiana & Arkansas Ry., has been promoted to be Chief Engineer and his former office has been abolished. He is a graduate of the University of Illinois, class of 1899, and was formerly with the engineering departments of the Peoria & Eastern Ry. and the St. Louis, Iron Mountain & Southern Ry.

J. S. McBride, formerly Valuation Engineer of the Chicago & Eastern Illinois R.R., has been promoted to be Principal Assistant Engineer with headquarters at Chicago. He is a

civil engineering graduate of Rose Polytechnic Institute. He has been continuously in the service of the Chicago & Eastern Illinois since 1905, when he was appointed an instrumentman on construction. He was promoted to be Assistant Engineer in 1908 and Valuation Engineer in 1914. He will continue to have supervision of the valuation work in addition to his new duties.

OBITUARY

William Cooper Cunts, Assoc. Am. Soc. C. E., General Manager and Treasurer of the Goldschmidt Thermit Co., New York City, died Nov. 2 at Auburndale, Mass. He was born in Hoboken, N. J., 45 years ago. He had been connected with the Goldschmidt Thermit Co. since 1910.

James C. Wallace, former President of the American Shipbuilding Co., of Cleveland, Ohio, died Oct. 31, aged 53 years. He started his business career as a machinist in 1881. Six years later he took charge of the drafting room of the Cleveland Shipbuilding Co., of which he became Vice-President and General Manager in 1893. When this company was merged into the American Shipbuilding Co., in 1899, he became General Manager, and in 1904 President.

ENGINEERING SOCIETIES

AMERICAN MINING CONGRESS.
Nov. 13. In Chicago, Secy., J. F. Callbreath, Munsey Bldg., Washington, D. C.

AERO CLUB OF AMERICA.
Nov. 13. Annual meeting in New York City. Secy., Wm. Hawley, 297 Madison Ave., New York City.

NATIONAL COMMERCIAL GAS ASSOCIATION.
Nov. 13-18. Convention in Atlantic City, N. J. Secy., Louis Stotz, 61 Broadway, New York City.

RICHMOND RAILROAD CLUB.
Nov. 13. Annual meeting. Secy., F. O. Robinson.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Nov. 14. Convention in Washington, D. C. Secy., W. H. Connolly, Washington.

SOCIETY OF NAVAL ARCHITECTS AND MARINE ENGINEERS.
Nov. 16-17. Annual meeting in New York City. Secy., D. H. Cox, 29 W. 39th St.

ELECTRIC POWER CLUB.
Nov. 15-17. Annual meeting in Hot Springs, Va. Secy., C. H. Roth, 1410 W. Adams St., Chicago.

KANSAS GOOD ROADS ASSOCIATION.
Nov. 16-17. Annual meeting in Lawrence Society headquarters in Kansas City.

SOUTHERN AND SOUTHWESTERN RAILWAY CLUB.
Nov. 16. Annual meeting in Atlanta, Ga. Secy., A. J. Merrill, P. O. Box 1205, Atlanta.

CITY MANAGERS' ASSOCIATION.
Nov. 21-23. Convention in Springfield, Mass. O. E. Carr, Niagara Falls, N. Y.

NATIONAL MUNICIPAL LEAGUE.
Nov. 23-25. Annual meeting in Springfield, Mass. Secy., C. R. Woodruff, 705 North American Building, Philadelphia.

RAILWAY GARDENING ASSOCIATION.
Dec. 5-7. Annual meeting in New Orleans. Secy., Charles E. Lowe, Sewickley, Penn.

PORTLAND CEMENT ASSOCIATION.
Dec. 11-13. Annual meeting in New York City. Assistant to General Manager, A. H. Ogle, Chicago, Ill.

The Northwestern Road Congress will meet for the third time, on Dec. 8 and 9 in Chicago. A tentative program has just been issued, and can be obtained from Secretary J. L. Long, Moline, Ill.

The American Society of Agricultural Engineers holds its tenth annual meeting at Chicago, Dec. 27 to 29. The preliminary program has been issued and can be obtained from the secretary, D. K. Shedd, Ames, Iowa.

A New York State Industrial Safety Congress is to be held Dec. 11 to 14, in Syracuse, N. Y. at the Hotel Onondaga. Addresses will be delivered on fire prevention, factory sanitation, etc. The New York City office is at 230 Fifth Avenue.

The North Carolina Drainage Association will hold its annual meeting Nov. 22 and 23 in Greensboro, N. C. This is said to be the first time in a number of years that the annual meeting of this organization has been held in an inland town or city.

The Engineers' Society of Western Pennsylvania has taken steps toward the purchase of "Solitude", the home of the late George Westinghouse, in Pittsburgh, and to convert the residence into a memorial. If the project is carried through, it will involve an expenditure of about $400,000.

The Detroit Association of the Members of the American Society of Civil Engineers was organized on Oct. 26, and officers were elected as follows: President, Theodore A. Leisen, General Superintendent Detroit Board of Water Commissioners; vice-presidents, Henry C. Ripley and Gale Cummins; secretary, Clarence W. Hubbell, consulting engineer, Detroit. Fifty charter members attended the organization meeting.

The Pittsburgh Association of Members of the American Society of Civil Engineers was organized on Nov. 4 at a meeting held in the rooms of the Engineers' Society of Western Pennsylvania, Pittsburgh. G. F. Davison and R. A. Cummings were continued as temporary chairman and secretary, respectively, and the committee, which reported on the advisability of forming the association, was continued and instructed to draft by-laws, and be ready to report at another meeting to be held in the near future.

American Society of Civil Engineers—The nominating committee has selected the following officers to be voted for at the annual meeting, Jan. 17, 1917: President, George H. Pegram, Chief Engineer of the Interborough Rapid Transit Co. and the Rapid Transit Construction Co., New York City; vice-presidents, George W. Kittredge, Chief Engineer, of the New York Central Lines, New York City; and George S. Webster, Director of the Department of Wharves, Docks and Ferries, Philadelphia, Penn.; treasurer, George W. Tillson, Consulting Engineer of the Borough of Brooklyn, New York City; directors, Alfred D. Flinn, Deputy Chief Engineer of the Board of Water-Supply, City of New York; Lewis D. Rights, Construction Manager of Lewis F. Shoemaker & Co., New York City; William R. Hill, Consulting Engineer, Albany, N. Y.; Arthur P. Davis, Chief Engineer of the United States Reclamation Service, Washington, D. C.; William L. Darling, recently Chief Engineer of the Northern Pacific Ry., St. Paul, Minn.; and R. H. Thomson, Consulting Engineer, Seattle, Wash.

The Association of Railway Electrical Engineers, which deals with train lighting and the general use of electricity for power and light in railway service, held its ninth annual meeting at the La Salle Hotel, Chicago, Oct. 31 to Nov. 3. The attendance was good and there was an extensive exhibit of railway and electrical appliances. In regard to train lighting, a committee submitted recommendations for standards; these have been considered also by the Master Car Builders' Association. Another committee reported on the system of lighting from a plant at the head end of the train, which system is in extensive use and is in general more economical than the individual car system. But where the service requires the frequent making up and breaking up of trains, and diverting cars to other lines, the individual equipment system is the only one that meets the requirement. Other reports related to the rating of lamps used in train service, and arrangement of lamps for lighting freight and repair yards. In relation to power and shop work, the reports dealt with the rating of crane and turntable motors, the use of compressed air in railway shops, and the use of electric welding for various purposes. It was pointed out that the saving in cost due to such welding has been so great that little attempt has been made at scientific management of the work. The time is approaching, however, when it will be necessary to reduce the cost of this electric welding work very materially.

Officers were elected as follows: President, C. J. Causland, Chief Electrician, Pennsylvania Lines; Secretary, J. A. Andreucetti, C. & N. W. Ry., Chicago.

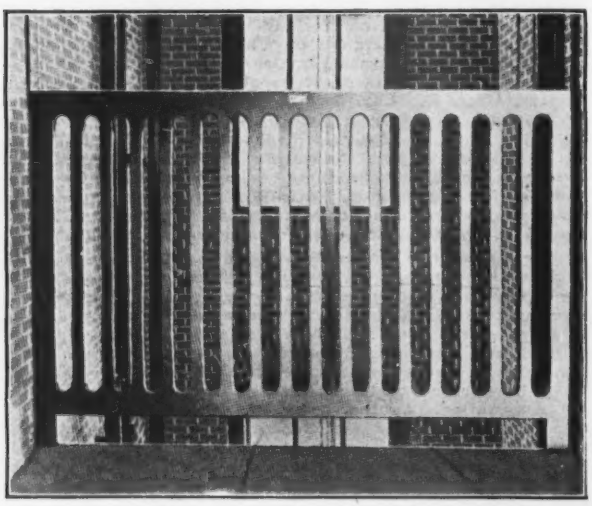
The American Road Builders Association—The annual business meeting of the Association for the election of officers was held in New York City at the Automobile Club of America on Nov. 3. The officers elected were as follows: President, A. W. Dean, Chief Engineer, Massachusetts Highway Commission; Vice-Presidents, Wm. H. Connell, Chief of the Bureau of Highways of Philadelphia; Austin B. Fletcher, State Highway Engineer of California; Prof. Arthur H. Blanchard, Professor of Highway Engineering, Columbia University; for Directors to serve three years, Paul D. Sargent, Chief Engineer of the Maine State Highway Commission; R. H. Gillespie, Chief Engineer of Sewers and Highways, Borough of the Bronx, New York City; T. R. Agg, Professor of Highway Engineering, Iowa State College; B. Michaud, Deputy Commissioner of Highways, Province of Quebec; Fred E. Ellis, Manager, Essex Trap Rock Co.; W. E. Atkinson, State Highway Engineer of Louisiana; Secretary, E. L. Powers, Editor of "Good Roads." At the dinner following the meeting, the retiring President, Col. E. A. Stevens, State Highway Commissioner of New Jersey, acted as toastmaster, and among the after-dinner speakers were Edwin Duffey, State Highway Commissioner of New York; and H. E. Breed, Deputy Commissioner of Highways of New York. Commissioner Duffey stated that under an act of the last legislature, the Commissioner, State Engineer, and Superintendent of Public Works, were given authority to prepare a code of rules for the control of commercial motor vehicles operated on high-

ways within the state with the object of protecting the roads from destruction. The commission has had an engineer at work for some months collecting the information on which to base such a code of rules and will shortly hold a public hearing upon them. They are to go into effect on Feb. 1 next.

Appliances and Materials

Pressed-Steel Elevator Gates

An all-steel freight-elevator gate with the principal member pressed from a single piece of sheet steel is being made by the Paterson-Leitch Co., of Cleveland, under the trade name "Steelite." The gate is as light as a wooden one and much more rigid; it costs but little more. After the main piece is pressed, the mitered corners are welded (oxyacetylene process), and whatever reinforcements are desired are spot-welded. An oak bumper is fastened to the bottom of each



ONE-PIECE PRESSED-STEEL ELEVATOR GATE

leg to cushion the impact and reduce noise. A two-section telescoping gate of similar design is made.

With these all-steel gates, are furnished steel-channel guides, steel weight boxes, ball-bearing pulleys, etc., for manual or semi-automatic operation.

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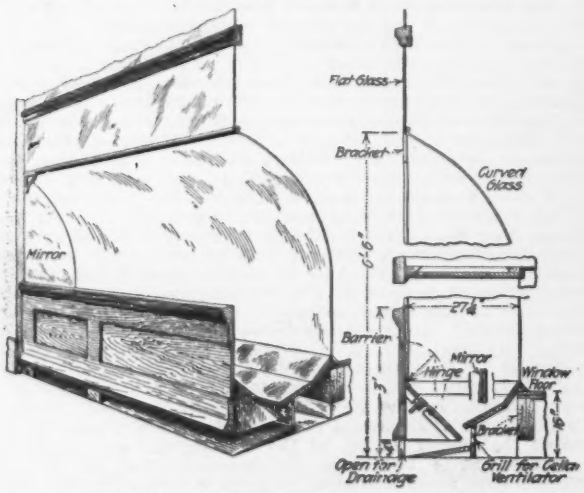
Non-Reflecting Store Window

A store front that clearly discloses the interior and eliminates the disturbing reflections commonly encountered has been developed by the Store Improvement Co., 1804 Finance Building, Philadelphia. This construction, shown in the accompanying sketch, is known as the "shadow-box window."



SHADOW-BOX STORE DISPLAY WINDOW

This result is produced by substituting for the usual flat plate glass two plates of glass the width of the window, having the upper, or "awning section" (above the line of vision), of ordinary flat glass and the lower, or "vision section," a bent glass plate set horizontally with the concave side toward the observer. The two plates meet in a plain clamp joint about a foot above a tall observer's eyes. A shadow box, or barrier, is then placed across the lower front of the window on the building line, with suitable provisions for drainage. This barrier may be of any material to harmonize with the building—as stone, iron, wood, kalamein, or iron or bronze grilles, etc.

In new buildings the installation of this type of window is easy and comparatively inexpensive—about the cost of good flat-plate construction. In old buildings the cost is more—and in some cases is prohibitive. Where a window is very wide, it is usually found best to divide it by mullions into two sections, or into a wide central section and two narrow end sections. Users of this window are obliged first to purchase a license and then to have the window constructed directly by the controlling company or by local contractors under the company's specications.

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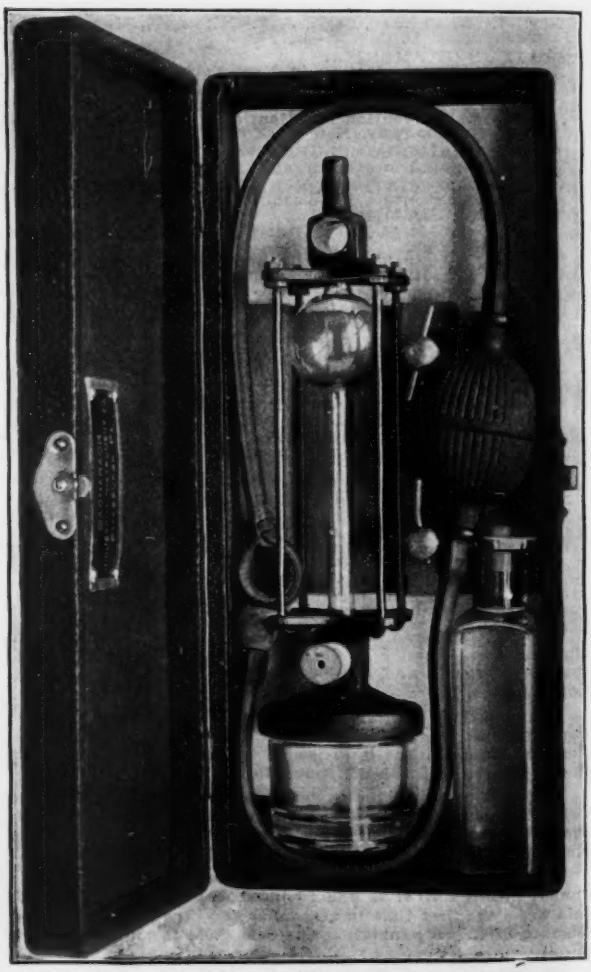
Pocket CO_2 Indicator with Replaceable Parts—A pocket CO_2 indicator for use by testing engineers, superintendents and power-plant operators has recently been brought out by the Bacharach Industrial Instrument Co., 14 Wood St., Pittsburgh, Penn. It works on the familiar principle of the CO_2 absorption by KOH. This instrument is designed for rather rough usage, and every part can be easily replaced in the event of breakage. It is reported that 17 to 20 analyses per



RUGGED CO_2 INDICATOR FOR QUICK TESTS

hour can be made with this instrument. With ordinary care an analysis can be made with a maximum error of 2%. The reagent is a solution of potassium hydroxide in water. Several hundred analyses can be made with one charge of the caustic. The device is compact and portable.